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PROCEEDINGS

PART II

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SOME RESEARCH PROBLEMS INVOLVED IN TRANSOCEANIC TELEPHONY

Edgar Marburg Lecture

By Frank B. Jewett[1]

It is an obvious fact that has become a commonplace to say that the world in the sphere of its human relations has become a very much smaller thing since the beginning of the nineteenth century than it was in all preceding time. While the coming of steam transport on land and sea was the first great step in this revolutionary change which has gone on progressively for more than a hundred years now, it was the advent of electrical communication which marked the greatest shrinkage in the world of interchange of men's ideas.

The invention of the electric telegraph and the coming of the submarine telegraph cable closed the door on one era and opened that to another. Little as it may have been appreciated at the time, the world of human relations was thereafter to be an entirely different thing from that which existed before. The influence of this new instrument which man, through science, had wrested from Nature's secret storehouse was destined to affect profoundly practically his every affair. No longer were great happenings in some remote part of the world to remain unknown to the people of other countries for days, weeks or even months. No longer was the representative of president, sovereign or prince to be an ambassador extraordinary and plenipotentiary in the old sense—a man of sagacity responsible single handed for his country's relations at a distant capital, whose acts were frequently set at naught by changes in home conditions of which he was totally unaware. He was to become in fact very largely the official messenger of his Secretary of State, making his every move as strictly in accordance with the current ideas of the latter as though space and time did not exist. On his part this same Secretary of State was to come to each day's work with a knowledge of world happenings as fresh as those of his own capital city.

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[1] Vice-President, American Telephone and Telegraph Co.; President, Bell Telephone Laboratories, Inc., New York City.

In the domain of commerce and trade almost every one of the rules and methods built up from the experience of untold centuries was to be swept aside and a new order of things instituted. No longer was the trading captain in a distant port to be the merchant adventurer who risked his reputation and the fortunes of his associates and himself on his judgment of what to buy and what price to pay for goods to be sold in the homelands months later. He was to know each morning the current condition of the home market and was destined in fact to obtain the monetary wherewithal for his needs, not from a strongbox in his cabin or by means of a cumbersome letter of credit, but in virtue of cabalistic dots and dashes transmitted at lightning speed from the exchequer of the home office.

In the realm of domestic affairs, forms of government were to undergo alteration. It was no longer to be safe for the clever and insincere politician to be all things to all men, with reliance placed on the laggard and inefficient qualities of communication.

Revolutionary as were the consequences which followed the advent of the electric telegraph, the really startling change came with the advent of the telephone. Bell's invention of only fifty years ago presaged a revolution in human relationships the like of which had never before been witnessed. To be able to converse by word of mouth with a friend or associate easily, at a distance and at any time, was to sweep space and time into the oblivion of the past and to bring the world of humans into the privacy of home or office at will. Gradually but inexorably and with increasing acceleration, the influence of telephony has been molding the form and substance of our existence. It has increased mightily the tendency of human beings to concentrate in urban communities. It has made possible the mammoth office buildings in which so much of business activity takes place, and it has introduced a vast new array of business methods which could hardly be conceived of as functioning except on the structure of the spoken word.

In the more recent past that development of telephony which we designate as "broadcasting" or as "public address systems" has still further accentuated the tendency to upset the old order of things.

Who can doubt that the facility provided by these instrumentalities, which enable a single human being to transmit his thoughts simultaneously and instantaneously to hundreds of thousands or millions of people, by means of the spoken word, will have a far-reaching effect in innumerable ways? In particular, may it not affect fundamentally both the form and substance of the thing we call government? May it not likewise influence the whole course of international relations by making it possible for men everywhere to know simultaneously and promptly the arguments which are advanced for or against proposed courses of action? Who can say but that the world may find it expedient to resurrect and re-employ forms of government like those of the Grecian city-states which it was once forced to discard because growing populations outran the range of the human voice?

For nearly fifty years the miraculous development of the telephone was confined to extension and betterment of communication over land areas. Even such minor water hazards as presented themselves in the course of this development were surmounted painfully because of the inherent difficulties of transmission. It came about thus that the intricate and extensive telephone networks were unique to the several continents. Ocean distances, which had for years been spanned by the submarine telegraph cable, were insurmountable barriers when it came to the transmission of speech.

With the advent of radio a new promise dawned and many wild and extravagant claims for an early and rapid interconnection of telephone systems by means of it were made. As is so frequently the case, however, imagination and prophecy far outran the possibility of actual achievement. It is only within the past two years that anything approaching a practical linkage of the separated continental telephone systems has come into being. Early in January 1927 the single transoceanic commercial telephone channel which the world now possesses was opened for the service of connecting telephone systems in and about New York City with those in and about London. Gradually, one step at a time, the service has been extended to enlarged areas on both sides of the Atlantic, until now every telephone station in the United States and a large part of Canada and Cuba has access to the transatlantic channel. On the European side, similarly, every telephone station in England and Scotland and a large part of the telephone stations in the principal centers of western Europe are in position to originate and receive calls to and from America.

While still far from the ultimate as regards efficiency, reliability or sufficiency of facilities, this single channel has already demonstrated conclusively that ability to reach and talk with people separated by ocean distances is something which the world desires and is willing to pay for. Nor is this desire measured in terms of a single element of human intercourse. The records of many hundreds of commercial calls which have been handled over the transatlantic link in the eighteen months since it was first put into service, show that they run the whole gamut from high finance to the trivial and frivolous. Bankers and brokers, statesmen, business men, fathers and mothers separated from home and family, sweethearts and friends, all have found occasion to use this newest of our agencies for eliminating distance and bringing people together for personal converse.

What the future has in store in the way of additional channels between Europe and America, or in the provision of corresponding channels to other continents, only time and experience can tell. The one thing now certain is that more and better channels are urgently needed on the transatlantic route, and one or more channels to South America. Thanks to the vagaries and limitations of radio and to the essential insufficiency of our present knowledge, the problem of providing these additional channels is one which will tax the ingenuity of science and engineering to the limit.

The story of commercial transoceanic telephony goes back some fourteen or fifteen years. Whatever people may have dreamed prior to that time, it is a fact that these dreams were for the time being mere speculation because of the lack of necessary physical tools. It was only after the advent of the vacuum tube that we had any real justification for believing that commercial telephony by radio over great distances was within the bounds of early possibility.

Invented by DeForest as a detector of radio telegraph signals, the vacuum tube, then called an audion, was developed in the laboratories of the Bell System into a stable, efficient and economical amplifier of voice frequency currents such as had to be coped with in making long distance wire telephony possible. Out of the knowledge derived from the research and development work attendant upon the production of vacuum tube amplifiers or repeaters came the realization that this same device in modified form might be made effective both for the production, modulation, amplification and detection of the currents necessarily involved in radio telephony. As soon as sufficient knowledge was acquired to justify the attempt, experimental radio telephony on a small scale was undertaken. Successful transmission of speech and successful interconnection of the ordinary wire telephone lines to radio telephone links was had in the early part of 1915, with transmission from Montauk Point, Long Island, to receiving stations in

New York; Wilmington, Delaware, and Brunswick, Georgia. As the latter distance was beginning to approach that involved in a transatlantic crossing, it was decided to make the attempt of a one-way transmission from America to Europe.

The story of the first successful transmission of speech between the new and the old world, and from Washington to far away Honolulu, has been told many times. It is a story replete with the romance of technical adventure and with the difficulties attendant on the accomplishment of anything in a period when the people of the world were locked in the life and death struggle of a world war. For the moment the one important fact is that an experiment was successfully performed in 1915 and intelligible human speech was transmitted from America to Europe. Like the flight of the Wrights at Kitty Hawk, it marked the beginning of a new epoch.

With the advent of the United States into the war, all attempts to follow up the initial experiments with the serious development of commercial transatlantic radio telephony had necessarily to be postponed. The period of the war and of the years immediately following were not, however, years of inactivity. Many of the problems involved in military communication had a direct bearing on the ultimate development of a successful long distance radio telephone service. In addition, the early experiments had indicated strongly the essential necessity of very great improvements in equipment, particularly in the vacuum tube itself, if a really satisfactory service was ever to come into being. The necessity for further extensive trials across the Atlantic was never lost sight of, however, and the temporarily postponed work was taken up again about 1920. By 1922 improvements in the vacuum tube, development of equipment, and the acquisition of more knowledge concerning long distance radio transmission made possible the trial of a one-way transmission having energies at the receiving end sufficiently large to insure adequate commercial service over the regular telephone networks employed for land telephony.

These trials of 1923 culminated in a demonstration lasting a number of hours, during which numerous people in New York spoke to a large number of distinguished scientists, engineers and men of affairs assembled in London. So sufficient was the volume of received energy that the voices of the New York speakers were delivered to the London listeners by means of loud speakers.

As there was at the time no transmitting station in England, the conversation was necessarily uni-directional, and was not therefore telephone service in the ordinary sense. It was sufficient, however, conclusively to prove the possibility of establishing two-way service at least for those times when radio transmission was sufficiently good to admit of operation.

Following the conclusion of this demonstration every energy, both in the United States and in England, was bent toward development, provision and trial of the equipment needed in the establishment of a two-way telephone connecting link. Despite all that had been done in the preceding years, very many intricate and troublesome problems still remained to be solved. By the early part of 1926, however, apparatus was in place on both sides of the Atlantic, and during the remainder of that year trials were continually under way. As was to be expected, many unforeseen difficulties arose and had to be overcome. These difficulties involved not only the physical things of equipment, but also the very scheme of operation originally indicated, and likewise the vexations of radio itself. Part of these vexations were inherent in the transmission of electro-magnetic waves through space, and part were involved in the chaotic conditions created by man in his employment of them.

Despite the never-ending succession of obstacles, real progress was made, so that in the fall of 1926 the more immediate practical problems of introducing the radio trunk between New York and London were ready for consideration. Surprising as it may seem, these problems in their way were almost as difficult of solution as were the physical problems which had preceded them.

Ordinarily we think of telephone service as being essentially the same wherever it is given. What actually developed when we finally came to grips with the matter was that only a single one of the essential factors involved in the giving of commercial long distance telephone service was the same in the United States and England. This single factor was the duration of the minimum conversation period to which the base rate was applied. Every other factor was different. All telephone calls in England were established on a desired number basis, while in the United States calling for particular parties had come to be the established method for very long distance service. The basis of charges beyond the initial period was entirely different in the two countries. The phrases employed by the operators in the establishment of connections and the handling of calls were dissimilar. Practices which had come to be a necessary part of the business in one country had no counterpart in the other, and so through the whole gamut of things.

Not the least difficult of the problems to be coped with was that involving differences in the currency systems of the two nations.

Obviously the charge for similar conversations originating on the two sides of the Atlantic had to be substantially the same, if equity was to prevail and objectionable practices avoided. Further, the base and overtime charges and the charges for special services had to be so adjusted as to be the same when reckoned in the fractional currency of the two countries.

Like the physical obstacles of development, these obstacles were finally overcome through cooperation between the American Telephone and Telegraph Co. and the British Post Office, and commercial service between New York and London, and their immediate environs, was initiated on January 7, 1927. At the time of the opening everyone involved had his own opinion as to what the future would disclose, but no one had any real knowledge of just what would develop. All the vast experience of fifty years gave no clue as to what this new thing was to mean in the realm of telephony. Were the inherent vagaries of the ether so great that stable transmission conditions could not be established? Was the community of interest across the Atlantic sufficient to develop more than an occasional demand for the service, or was it so great as to prove an embarrassment once the service was known to exist? Were the inherent costs of this type of telephone channel so great as to be beyond the ability of the telephone using public to pay? All of these questions and many others of a like kind were discussed and argued, but always in the last analysis against a background of substantial ignorance.

Time and experience have provided partial answers to many of the questions. We know that, taken by and large, the service despite its deficiencies is sufficiently reliable and satisfactory to be used. We know that without any endeavor seriously to advertise or develop the service the number of offered calls is already in excess of the facilities. Unfortunately we know also that the problem of providing additional facilities is even more difficult than we had originally anticipated. Nevertheless we have every reason to believe that a sufficient number of channels of fair reliability can be obtained in due course.

A word now about the extent to which the service has been employed since it was opened to the public, the classes of users and the distribution of calls as between the different countries served.

It was inevitable that the introduction of anything so new and revolutionary as the establishment of a speech channel across the Atlantic should for a time develop a considerable amount of use by the merely curious. How long this period of curiosity calling persisted we have no means of knowing accurately. From the beginning, however, interspersed with the calls of the curious, were calls by those who had real business or social reasons for wishing to avail themselves of the facility. After the period of curiosity calling there were, during the spring and summer of 1927, a number of months when the use of the circuit was but a few calls a day. Gradually, however, people came to know and rely on the service and, beginning last fall, there was a steady growth in offered business, until the number of calls handled on many days far exceeded the number of calls handled over long distance wire lines, where good commercial service is provided. The only way in which this could be done was by incurring excessive delays between initiation and completion of the calls. While such a practice has been standard in some foreign telephone services, it is very far from the standards aimed at in the United States. With but a single circuit physically available, there has been, however, no other way of serving the telephone-using public desiring transatlantic connections. The provision of additional channels will bring the service more nearly in line with that which the public has come to expect over land lines.

From the opening of service on January 7, 1927, to the close of business on June 16, 1928, a total of 6368 commercial messages have been handled over the transoceanic link. When it is remembered that the difference in time between America and Europe limits the telephone day to a very few hours, this record of 12.1 messages per day for a service so new, strange and relatively expensive, is rather surprising. Compared with this average for the entire period since service was inaugurated, the average daily calling rate since April 1, 1928, has been 36.5 and the largest day's traffic has been 61 calls.

Of the total messages handled 25.2 per cent appear to have been between bankers and brokers, 0.9 per cent between theatrical people, 1.5 per cent between merchants, 1.3 per cent between representatives of the press, 43.0 per cent have been social calls, and 28.1 per cent miscellaneous.

In the matter of distribution of messages as between countries, but counting the United States, Canada and Cuba as a single unit, 77.3 per cent have been to England, 15.1 per cent to France (Paris), 5.0 per cent to Germany, 1.5 per cent to Holland, and 1.1 per cent to Belgium and Sweden. This distribution is somewhat misleading, however, because of the fact that for somewhat more than the first year that the channel was in operation there was no extension to continental European countries. Since April 1, 1928, the distribution of messages has been 58.2 per cent to Great Britain, 31.0 per cent to France, 6.6 per cent to Germany, 2.4 per cent to Holland, and 1.8 per cent to Belgium and Sweden.

As most of you doubtless know, the radio portion of the transatlantic link consists of two parts, an eastbound channel with a radio transmitter at Rocky Point, Long Island, and a receiving station at Cupar, Scotland; and a westbound channel with a transmitter at Rugby, England, and a receiving station just outside of Houlton, Maine. While differing somewhat in the details of arrangement and construction, the two transmitting stations and the two receiving stations are functionally alike. Despite the fact that the east and westbound transmissions are over different routes, both transmissions are on the same wave length. Originally separate wave lengths were experimented with and while the problem is technically much simpler when separate wave lengths are used, the crowded condition of the ether impelled us to seek the present solution as a means of conserving channel space.

The location of the receiving stations at points widely separated from their associated transmitters and to the north of them, is due both to the necessity for limiting the amount of energy picked up by the receiving antenna from its own transmitting station, and to the necessity of locating the receiving station as far north as feasible because of the better average atmospheric conditions in northern latitudes. Connection to and from the transmitting and receiving stations is made over high-grade loaded and repeatered telephone lines, and connection to the normal telephone network is made at the telephone switchboards in New York City and London where the lines of the transmitting and receiving stations come together. Calls from any point in America for a place in Europe are routed into the long distance board in New York City as they would be if destined for the station of a subscriber in New York City. They are then routed over the peculiar radio channel to the long distance switchboard in London, where in turn they are connected to the normal toll lines for points in Great Britain or on the continent. While the peculiarities of the radio channel require the presence of numerous people not normally involved in a wire connection, and necessitate entirely special operating methods, the establishment of a call from America to Europe is not at all different from the establishment of any other long distance call so far as the subscriber is concerned. Nor is the subscriber normally cognizant of the fact that a radio link is involved in the connection except when static disturbing conditions prevail.

As neither the appropriateness of this occasion nor time calls for or would permit any detailed engineering description of the transatlantic link, and as this audience is presumably interested largely in some of the more material problems with which we have had to cope, I propose in the time which remains to outline to you some of the more important steps which we have had to take.

Looking over the history of this development there appear to be three or four problems which in magnitude and importance are dominant in character. Among these have been the development of the vacuum tubes for handling large amounts of energy at the transmitting station; the development of a system and apparatus to permit of ordinary two-way telephony over a radio system employing the same wave length in both directions; the development of antenna structures at the receiving stations for maximum efficiency as regards the particular wave length employed, and of minimum efficiency as regards all other electromagnetic waves, whether produced by atmospheric disturbances or by other radio stations; and the development of suitable transmitting, receiving and line terminating equipment to provide, from beginning to end of the link, a communication channel of such a character that it can be handled substantially as an ordinary toll line in the daily operation of the plant.

Possibly the development of the high-powered vacuum tubes employed in the transmitting stations involved the greatest amount of technical difficulty and necessitated the greatest amount of research and development work of anything concerned with the solution of the entire problem.

The power tubes employed in the original transatlantic experiments of 1915 were essentially enlarged forms of the vacuum tube as it had then been developed for telephone repeaters or for use in radio telegraph receiving sets. They were air-cooled, essentially low voltage devices of very limited power output, and were constructed of materials easily damaged by the excessive temperatures of electron bombardment. Because of their limited unit power capacity there was only one way in which even a moderate amount of high-frequency energy could be delivered to the transmitting antenna, namely, by employing a large number of tubes in parallel. The transmitting station at Arlington in 1915 was equipped with about 500 of these tubes, each having a normal rating of about 25 watts. The transmitter was anything but commercial as the arrangement of so many tubes provided innumerable opportunities for serious trouble which very frequently resulted in the destruction of one or more tubes.

It was clearly evident at the conclusion of the 1915 experiments that if we were ever to have satisfactory radio telephone transmission, where satisfaction was measured in the terms normally applicable to wire telephony, it was imperative that vacuum tubes of enormously increased capacity must be developed. Contrasting with the state of the art in 1915 is the present situation in which the transmitters at Rocky Point and Rugby are equipped with fifty tubes each of about 10 kilowatts power rating.

Unlike the vacuum tubes of the earlier art, these modern tubes, while functionally the same, are structurally different. They are water rather than air-cooled—they employ enormously high voltages—the plate of the three-member audion forms part of the outer casing of the vacuum chamber instead of being enclosed within it, and the materials used in the construction of the tubes are different. Oddly enough, these tubes, with output rated in kilowatts, are not very much larger in physical dimensions than were their 25-watt progenitors thirteen years ago.

The development of water-cooled tubes was made possible through an invention of Mr. E. R. Stoekle sometime before the 1915 experiments. This invention consisted of making the anode of the tube a part of its envelope and having water circulated about it in order to conduct away the energy loss of the tube. Stoekle's invention, however, could not be well utilized in a commercial way until, as a result of much research work, Mr. W. G. Houskeeper in 1917 developed a method of obtaining a vacuum-tight seal between glass and a metal whose temperature coefficient of expansion was different from that of the glass. By employing these two inventions vacuum tubes were developed in which the anode was made of a copper cup sealed to the glass of the envelope by means of a so-called Houskeeper seal. The present-day tubes used at Rugby and Rocky Point are constructed of copper-sealed-to-glass for the anode and containing envelope, tungsten for the filament, and molybdenum and nickel for the inside parts, properly mounted on and insulated by special insulating materials, usually lavite or isolantite.

In the earlier forms of the high-power tubes relatively low-melting point glass was employed. More recently, however, it has been found possible to make the Houskeeper seal to pyrex glass, and tubes of this construction are used in the final designs for both long and short-wave transmitters.

The control of the properties of the materials employed in vacuum tube construction is of the utmost importance in the manufacture of satisfactory tubes. Minute variations in composition or characteristics may easily render the tubes unuseable. The minutest leak in the enclosing envelope is of course fatal, as is also anything which tends to produce an incipient arc discharge within the tube when operating under the thousands of volts employed between plate and filament.

Power tubes of much larger rated capacity than the 10-kilowatt tubes now employed have been constructed and it may well be that the ultimate high-power transmitter for long-wave operation will employ but a single or, at most, a very few tubes of capacity rated in hundreds rather than in tens of kilowatts. The decision involves both technical and economic questions. There are some obvious advantages to the elimination of parallel operation of vacuum tubes, while on the other hand there are very serious technical and cost questions attendant upon the manufacture and operation of large size tubes employing twenty thousand or more volts. In the final decision probably the factors of reliability and uniformity of operation will have more weight than questions of initial or operating cost.

In the matter of the system of transmission to be employed, the scheme now in use is the result of a progressive development of the early arrangements of 1914 and 1915. In the earlier forms of radio telephony the carrier current modulated by the voice frequency currents and amplified in the final stages of the transmitter was delivered to the antenna to be sent into the air in the form of three separate radiations; (1) a radiation of energy of the carrier itself, (2) a complex radiation of the carrier frequency plus the voice frequencies, and (3) a complex radiation of the carrier frequency minus the voice frequencies. The two latter components are known as side bands. This arrangement when applied to high-power transmission is fraught with very serious limitations, not only in the design of the antenna but in the strength of the effective signal and in the matter of antenna insulation.

The circuit employed on the long-wave transatlantic circuit radiates but one of the side bands. This enables a much greater effective signal to be transmitted than is possible with the use of the ordinary carrier and double side band arrangement, and likewise admits of the use of a sharply tuned antenna. In obtaining the single side band a carrier of 33,000 cycles is first modulated by the voice currents and one of the side bands is picked out, by means of filters, which can be accurately designed at these frequencies. A second carrier of approximately 90,000 cycles per second is then modulated with this side band. This second modulation in turn produces side bands of approximately 60,000 cycles and 120,000 cycles. The 60,000-cycle band is filtered out and used in the final transmission. The employment of this system not only enables effective filtering to be obtained but, on account of the large frequency differences between carrier and side bands in the second stage, permits of considerable adjustment of the frequency of transmission to be made, using fixed filters.

The transmitting antenna is of the multiple tuned type and has a characteristic which is too narrow for telephonic uses. The band is broadened to the necessary extent by the use of a special coupled circuit intermediate between the amplifier and the antenna.

In the arrangements used at Rocky Point and Rugby, the final amplifier in which the real energy to be radiated is supplied consists of thirty 10-kilowatt tubes operating in parallel. The use of this number of tubes is accompanied by considerable difficulty due to the oscillations set up between one tube and another. Special precautions have to be taken to eliminate this source of disturbance.

When we came to consider the possibility of using the same frequency of carrier for transmission in both directions, we were confronted with a difficulty not normally present in ordinary wire telephony. Every ordinary telephone circuit used for two-way conversation has its terminal apparatus arranged on a duplex or balanced basis. When so arranged only a small part of the outgoing energy is returned to the receiving side of the circuit. This kind of an arrangement is satisfactory in ordinary telephony and telegraphy because of the fact that while the differences between the current at the transmitting and receiving ends of the circuit are great, the differences are not so excessive that they cannot be coped with in the design of apparatus. When, however, we come to deal with long distance radio telephony, the discrepancy is so great as to be entirely beyond the bounds of mechanical and electrical accuracy in the design of balancing apparatus. In this case we are dealing with transmitter energies measured by hundreds of horse power and with receiving energies almost infinitesimally small. Any attempt to operate such an unbalanced system in two-way fashion would result in returning to the receiving side more energy from the local transmitter than is normally received from the distant transmitter, and the circuit would consequently produce a sustained oscillation. If therefore the same radio channel is to be employed in duplex operation an entirely new scheme must obtain.

The solution arrived at finally was what has been termed "voice frequency switching." In this arrangement the whole circuit is alternately under the control of the currents produced by the two speakers. When speaker "A" is talking, instantaneous vacuum tube switches operate to prevent any energy from his transmitter being returned through the local radio receiver. When speaker "B" talks, similar relays at his end of the line are put in operation. While therefore the radio link is a duplex channel so far as the wire lines or the speakers are concerned, it actually operates as a unilateral transmission channel which changes its direction of transmission to correspond to the one of the speakers who is talking.

A well-known deficiency of all radio reception is the presence at times in the receiver of disturbing electromotive forces produced either by radio stations other than the one desired to be heard or more frequently by atmospheric disturbances commonly known as "static." This deficiency, objectionable as it is in broadcast receivers, is an even more serious obstacle in a system employed in commercial telephony.

Although disturbing electromotive forces cannot be entirely eliminated, their effect can be minimized by certain forms of receiving antennæ. Fortunately also, these forms of antennæ when properly installed, tend to strengthen the signal from the distant transmitting station with which they are associated. In the arrangements used at Houlton and Cupar, what are known as wave antennæ are employed. These wave antennæ are in effect a series of carefully constructed straight parallel open-wire telephone lines a number of miles long, so oriented as to have maximum efficiency of reception for electromagnetic waves coming along the great circle direction from the distant transmitting station. Such an antenna system, having markedly less efficiency of pick-up for transmissions in other directions, has a high degree of selectivity. It is strongly affected in deleterious fashion only when the source of static disturbance or the disturbing transmitter is substantially in the same direction as the station it is designed to receive from. Fortunately, at Houlton, Maine, the majority of static disturbances appear to come from regions to the southwest, which is the direction of minimum efficiency of pick-up of the wave antenna.

What I have described thus far are certain features of the long-wave channel, which is the one now regularly employed in the transatlantic link. In addition, we have been experimenting with and are using somewhat, an entirely different arrangement employing very short electromagnetic waves, that is, waves of from 15 to 30 meters. The high frequencies to which these short wave lengths correspond are apparently subject to very different laws of transmission than those involved with long waves. The facts about short wave transmission are still quite meagre, although there is prospect of obtaining satisfactory telephone channels by their use. As a matter of fact, if any considerable number of transoceanic telephone channels are required and have to be provided by radio, they must be looked for primarily in the short end of the radio spectrum.

Thus far the high frequencies required for short-wave transmission have been produced by successively selecting and amplifying the harmonics obtained from an oscillating vacuum tube source of approximately 2.2 megacycles, the actual frequencies used in our present short-wave transatlantic circuit being about 18.3 megacycles. The type of transmission employed is the ordinary arrangement of transmitting both the carrier and the two side bands.

With short-wave transmission it is necessary to employ special means to obtain constant frequencies. These means involve frequency control by a quartz crystal whose temperature is kept approximately constant by a sensitive thermostatic control. The oscillator system consists of two 250-watt tubes and modulation is secured by the usual plate modulation methods.

The high-power amplifier which feeds the antenna consists of two stages, both employing water-cooled tubes, the final stage having two tubes operating on a push-pull arrangement and delivering about ten kilowatts to the antenna. These tubes are specially constructed for high-frequency work, the filament and grid leads being brought out at opposite ends of the tube. The whole question of tube and apparatus construction for high-frequency systems is one of extreme intricacy due to the relatively large capacity effects which obtain.

Unlike the transmitting antenna for long-wave working, where enormously high and extended structures have to be erected, the transmitting antenna for short-wave operations is quite simple as to size though somewhat complex as to form. No final form of structure has yet been determined upon. The antennæ at present in use and directional in effect consist of arrays of half wave radiators fed by a special system of transmission lines. Each short wave transmitter is associated with three arrays designed for three different wave lengths which are made necessary on account of diurnal and seasonal variations of transmission in the range of wave lengths employed.

At the short-wave receiving station the antennæ are also of a directional character designed to give high pick-up in the direction from which the signal is coming and to give low pick-up in the opposite direction. Just as in the case of the long wave, this latter property is of importance in keeping down extraneous interference.

The receiving set proper now in use is of the double detection variety and has a volume control circuit which maintains the signal at approximately the same level, irrespective of changes in strength of received signal. This latter feature is of the utmost importance here, since all experience with short waves indicates that the transmission is subject to wide and erratic fluctuations known as "fading." So pronounced is this characteristic that without some form of regulator commercial telephony would be quite out of the question. Even with the regulator, it now appears likely that transmission will be uncommercial over any wave length channel a goodly portion of the time.

If time permitted I might tell you numerous interesting results of our research work in fields collateral to and connected with the giving of transatlantic telephone service, and of the numerous materials which have had to be developed. I have already trespassed too long, however, and in closing would merely state that the very remarkable magnetic alloy of nickel and iron, which we know as "permalloy," has been a most important factor in these other developments. Not only is it used in all of the loading coils employed on the long distance telephone lines, but in much of the apparatus involving magnetic materials as well. Beyond this I might mention merely the construction of the filament employed in all the smaller vacuum tubes, such as those required for telephone repeaters and in the radio receiving apparatus. This filament, unlike that employed in the high-power tubes, which is of drawn tungsten, is what is known as "Wehnelt" cathode. In these smaller tubes the filament consists of a ribbon or wire core of a platinum-nickel alloy coated with the oxides of barium and strontium. To obtain this filament it has been necessary to purify the platinum to a purity equal to or greater than that of the highest purity platinum made especially for standard thermocouple junctions. The preparation of this platinum-nickel alloy in the large quantities required has been made possible through studies of refractories capable of withstanding the high temperatures at which this alloy melts. The nickel required both for filament alloy and for anode and grid structures, and the glass and insulating materials which are employed, require special and elaborate treatment to obviate any chance of deleterious gas emission during operation or the production of secondary ionization effects.

As a final word preliminary to showing you in pictorial form the establishment of a long distance connection involving wire telephony and the transatlantic link, and to give you some idea of the numerous transformations and revivifications without distortion, to which the telephone currents are subjected, I would state that in a connection by wire and radio from San Francisco to London more than 600 vacuum tubes for repeaters, oscillators and high-power radio amplifiers are involved.

[The Lecture was concluded with the showing of a motion picture illustrating the operations involved in establishing a telephonic connection across the Atlantic Ocean.—ED.]

FERROUS SILICATES IN STEEL[1]

By C. H. Herty, Jr.,[2] and G. R. Fitterer[3]

Synopsis

The paper represents the results of an investigation on deoxidation with silicon, with particular emphasis on the type of inclusion formed when various amounts of silicon are added to a bath of steel and on the effect of the silicates on the forging and rolling of the steel. Various types of silicates formed on deoxidation are discussed with reference to their melting points, micro-structure and coalescence in the steel. From a consideration of certain equilibria the type of silicate which will be formed on deoxidation with silicon in any grade of steel in the open hearth may be predicted—this in the absence of manganese added simultaneously. The effect of manganese on the silicates so formed is discussed. The experimental ingots were forged and the type of silicate was found to have a decided effect on the forging of the steel. It is shown that in order to predict the types of inclusions which will be formed under any conditions of deoxidation, an extended study of the three deoxidizers, silicon, manganese and aluminum, is necessary.

Introduction

Scope of Report

The object of this study was to obtain quantitative information on the types of silicates obtained when steel is deoxidized with silicon; to obtain information on the rate of coalescence and segregation of the various types of inclusions; to determine the effect of the silicates on the forging properties of steel and conversely the effect of forging on the inclusions; and to determine the equilibrium conditions between iron, silicon, and oxygen and their relation to deoxidation with silicon.

This paper presents data on the types of silicates formed on deoxidation with silicon, the segregation and relative rates of coalescence of the various types of inclusions, and the forging of the steels containing the inclusions.

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[2] Physical Chemist, U. S. Bureau of Mines Experiment Station, Pittsburgh, Pa.

[3] Junior Metallurgist, U. S. Bureau of Mines Experiment Station, Pittsburgh, Pa.; formerly Research Fellow, Carnegie Institute of Technology.

EFFECT OF IMPURITIES ON STEEL

From a review of the literature on the effect of oxides and sulfides in steel, it is evident that these impurities may be held responsible for internal notches, low transverse strength and ductility, fatigue failure, etc., and in many instances corrosion of steel has been traced to the presence of oxides and sulfides. They may also, if present in sufficient amounts, cause red-shortness in rolling and surface defects, particularly in steels such as plates and full finished sheets.

A complete review of the literature on this subject is too lengthy for this paper. Some of the outstanding workers in this field have been Law,[1] Howorth,[2] McCance,[3] Levy,[4] Matveieff,[5] and Dickenson.[6] Lucas[7] has shown that the path of fatigue failure in a specimen of Armco iron was due primarily to non-metallic inclusions. Many instances are cited in the discussion of this paper where cracking of various steels could be traced to the presence of slag inclusions. Oxides in steel are discussed at considerable length by Hoyt.[8]

It is unquestionably true that certain types of inclusions may be very harmful in steel, whereas others may be essentially harmless. In commercial steels it is difficult to determine the exact composition of many of the inclusions, particularly if the steel is "rimmed" rather than killed. It has, therefore, seemed advisable to make a quantitative study of each silicate and sulfide when there is definitely only one present, and from these studies to draw conclusions of practical value and to point the way to further work on combinations of the various silicates and sulfides.

DEOXIDATION PRODUCTS OF SILICON

The first product of deoxidation with silicon is silica, as shown by the following reaction:

$$Si + 2FeO \rightleftarrows SiO_2 + 2Fe$$

If excess iron oxide is present the silica may flux with this, forming low-melting ferrous silicates. Thus with high-carbon steels, which would necessarily be low in iron oxide, silica would persist as the predominating type of inclusion; whereas with low-carbon steels the predominating form would be ferrous silicate, unless the silicon addition were too large. As is shown later, manganese oxide has a great effect on the type of inclusion to be expected under operating conditions, and its effect must be considered in applying the results of the present study.

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[1] *Journal;* Iron and Steel Inst., Vol. 69, p. 134 (1906); *Journal,* Iron and Steel Inst., No. 2, p. 94 (1907).

[2] *Journal,* Iron and Steel Inst., No. 2, p. 301 (1905).

[3] *Journal,* Iron and Steel Inst., Vol. 97, No. 1, p. 239 (1918).

[4] *Journal,* Iron and Steel Inst., Carnegie Scholarship Memoirs, Vol. 3, p. 260 (1911).

[5] *Revue de Metallurgie Memoirs,* Vol. 17, p. 736 (1920).

[6] *Journal,* Iron and Steel Inst., Vol. 113, No. 1, pp. 177–196 (1926).

[7] *Transactions,* Am. Soc. Steel Treating, Vol. 11, p. 531 (1927).

Metallography, pp. 261–280 (1921).

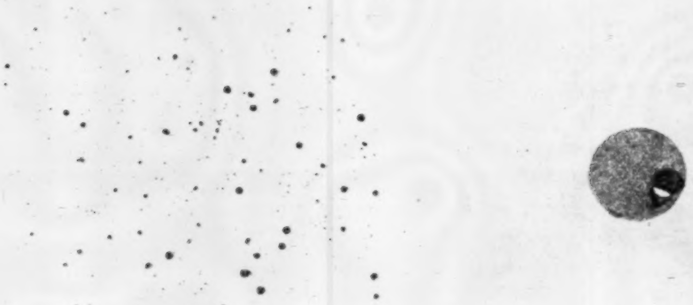
Method of Attack:

In order to determine the types and properties of inclusions formed upon deoxidation with silicon it was necessary that four conditions be fulfilled: (1) the silicates must be frozen in the steel quickly enough so that the minimum elimination would take place; (2) the steel should contain enough silicate inclusions so that a complete metallographic and chemical examination could be made; (3) the silicate inclusions formed must be essentially iron silicates with as small amounts of impurities such as manganese oxide, alumina, etc., as possible; and (4) the ingots must be large enough to allow comparison with industrial ingots as to physical action in the mold and segregation in the ingot.

The method of attack decided upon was to melt down 250 to 300 lb. of boiler punchings and 2 lb. of crushed coke in a Moore 'Lectromelt furnace. A slag of lime, iron oxide and spar was put on the metal as the melting progressed, and the heat was ored down to 0.04 to 0.06 per cent of carbon. At this time the metal contained approximately 0.02 to 0.04 per cent of manganese, 0.006 to 0.010 per cent of phosphorus, 0.025 to 0.035 per cent of sulfur, and 0.005 per cent of silicon. The analysis of the finishing slag was: SiO_2, 9 per cent; P_2O_5, 0.8; FeO, 27.5; Fe_2O_3, 11; MnO, 3; CaO, 42; MgO, 4; and total sulfur, 0.48 per cent; Al_2O_3, not determined.

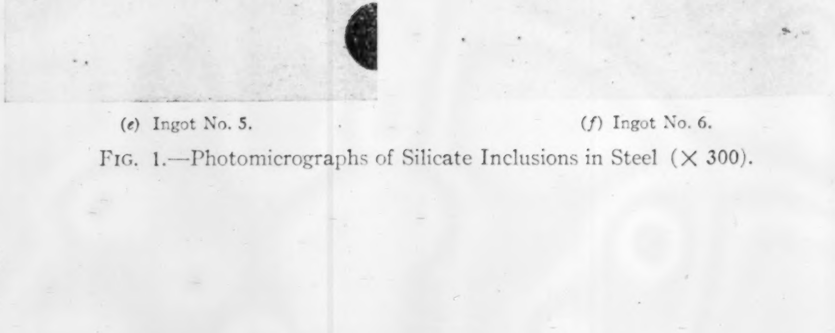
When the heat was ready to be tapped, 15 to 25-lb. ladles of metal were poured, the deoxidizer (50-per-cent ferro-silicon) being added in the ladle. As soon as the deoxidizer had melted, the metal was poured immediately into 3-in. square ingot molds, and solidification took place in a few minutes. The total time from addition of deoxidizer to pouring into the mold was about 30 to 40 seconds in every case.

The ingots were split longitudinally and samples cut at nine points, as shown in Fig. 4, for the determination of non-metallic inclusions. Corresponding samples were cut for microscopic examination, and in some cases 18 drillings were taken from the ingot to determine segregation of metalloids. The silicates in the steel were determined by the Dickenson method, which consists of dissolving the steel in cold 10-per-cent nitric acid followed by a suitable treatment of the residue.



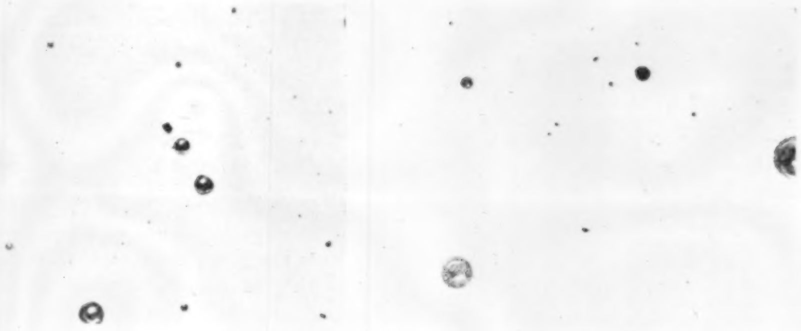
(*a*) Ingot No. 1. (*b*) Ingot No. 2.

(*c*) Ingot No. 3. (*d*) Ingot No. 4.

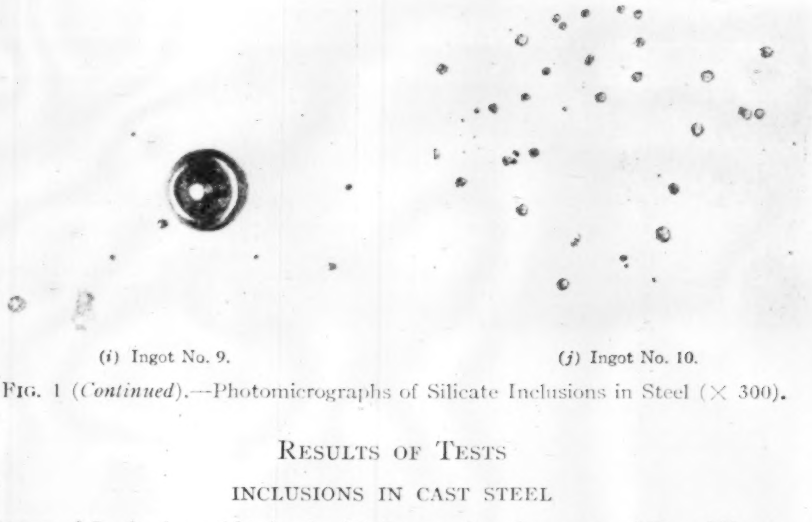


(*e*) Ingot No. 5. (*f*) Ingot No. 6.

FIG. 1.—Photomicrographs of Silicate Inclusions in Steel (× 300).



(g) Ingot No. 7. (h) Ingot No. 8.



(i) Ingot No. 9. (j) Ingot No. 10.

FIG. 1 (*Continued*).—Photomicrographs of Silicate Inclusions in Steel (× 300).

RESULTS OF TESTS

INCLUSIONS IN CAST STEEL

Types of Inclusions Obtained with Increasing Amounts of Deoxidizer:

Figure 1 and Table I give the results on a typical heat. On this heat, 10 ingots were poured, the photomicrographs reproduced in Fig. 1 showing the inclusions in the cast steel from each ingot. Figure 1(*a*) shows the iron oxide in the original steel, an ingot poured with no deoxidizer added. The small rounded inclusions are typical of FeO in steel; and the FeO content of this ingot was equal to 0.075 per cent of oxygen. Figures 1(*b*), (*c*) and (*d*) show ingots with increasing amounts of silicon added in the ladle, and it will be noted that large dark gray to black ferrous silicates are formed, also that as the silicon addition increases, the number of small iron oxide spots decreases. Figure 1(*f*) shows the inclusions in the sixth ingot which was rimmed. It is interesting to note in this photomicrograph that two of the inclusions are of the dark gray type; the larger inclusion has a duplex structure, and the third large inclusion has begun to take on a glassy aspect typical of high-silica silicates in steel. This is referred to hereinafter as the transition type of inclusion in going from high-iron silicates to high-silica silicates. Figures 1*(g)*, *(h)* and *(i)* are the typical glassy type of inclusion found when steel is killed with silicon. These inclusions analyzed approximately 75 to 95 per cent of SiO_2, and the ring and dot appearance is due to reflection of light from the steel surface under the glassy silicate particle. Figure 1*(j)* is extremely interesting in that this ingot resulted from the deoxidation of the last lot of metal in the furnace—amounting to about 3 lb. To this metal was added 100 g. of ferro-silicon, giving a silicon content of about 2 per cent. The small amount of metal in the ladle would not take up all the ferro-silicon, and the metal was pasty when poured into the mold. The total time between the silicon addition and solidification of the metal was only about 45 seconds; and the silica inclusions contained in the metal were small and of the high-silica type. If this photomicrograph be contrasted with Fig. 1*(i)*, the difference in size of particle may readily be seen. The difference in particle size is entirely due to the lack of time for coalescence of the high-silica inclusions.

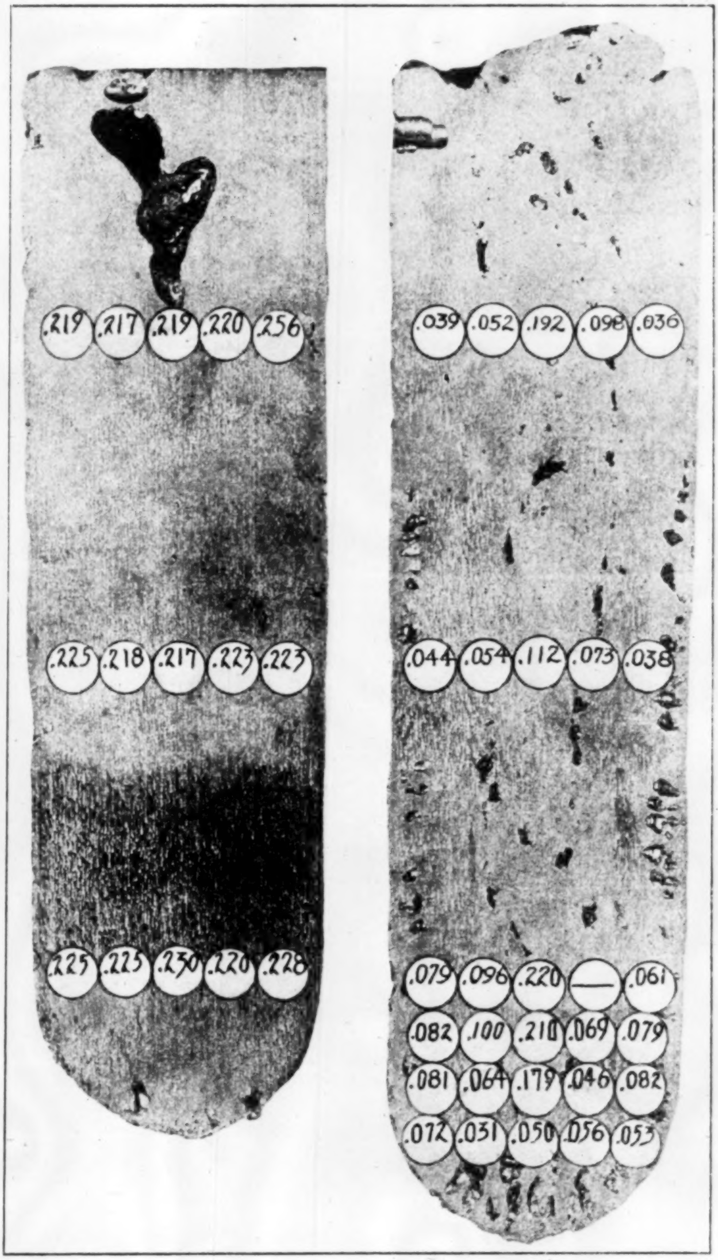
TABLE I.—TABULATED RESULTS ON DEOXIDATION OF A HEAT OF LOW-CARBON STEEL WITH SILICON.

Ingot	Photo-micrograph	Amount of 50-per-cent Ferro-Silicon Added to Average 20-lb. Ingot, g.	Analysis of Steel, per cent			Approximate Analysis of Inclusions, per cent	
			Carbon	Total Silicon	FeO expressed as per cent of oxygen	FeO	SiO_2
No. 1	Fig. 1*(a)*	0	0.046	0.004	0.075	100	0
No. 2	Fig. 1*(b)*	5	0.048	0.005	0.052	90	10
No. 3	Fig. 1*(c)*	10	0.035	0.020	0.0675	70	30
No. 4	Fig. 1*(d)*	15	0.038	0.021	0.039	70	30
No. 5	Fig. 1*(e)*	20	0.054	0.040	0.0338	50	50
No. 6	Fig. 1*(f)*	30	0.069	0.050	0.0207	35	65
No. 7	Fig. 1*(g)*	40	0.069	0.097	0.0128	30	75
No. 8	Fig. 1*(h)*	50	0.044	0.120	0.0083	20	85
No. 9	Fig. 1*(i)*	75	0.048	0.212	0.0111	5	95
No. 10	Fig. 1*(j)*	100 (3-lb. ingot)	0.055	1.77	......	1	99

SEGREGATION

Segregation of Silicon:

The surfaces of the split ingots No. 6 and No. 9 were drilled in the various regions indicated in Fig. 2. The samples were analyzed


.219 .217 .219 .220 .256
.225 .218 .217 .223 .223
.225 .225 .230 .220 .228
.039 .052 .192 .098 .036
.044 .054 .112 .073 .038
.079 .096 .220 — .061
.082 .100 .210 .069 .079
.081 .064 .179 .046 .082
.072 .031 .050 .056 .053

(*a*) Ingot No. 9 (Killed Steel). (*b*) Ingot No. 6 (Rimmed Steel)

FIG. 2.—Segregation of Silicon in Rimmed and Killed Steels.

for total silicon and their respective percentages are represented on the figure.

In the killed ingot (ingot No. 9) little segregation of silicon can be observed, since with one exception all samples analyze between 0.217 and 0.230 per cent.

Segregation in the rimmed steel (ingot No. 6) is high and is indicated as increasing toward the vertical center line of the ingot with a maximum toward the bottom of this line as represented by the point designated 0.220 (representing percentage of silicon). If the bottom twenty analyses are considered more in detail it will be observed that the segregation of silicon can be represented as a pyramid with its apex at the sample containing 0.220 per cent of silicon. The sides of the pyramid can be seen by considering the twenty analyses as arranged in five vertical columns each containing four analyses. Starting from the left to the right, the highest analysis in the first row is 0.081 or 0.082 per cent; in the second row, 0.100 per cent; in the third, 0.220 per cent; in the fourth, 0.069 per cent; and in the fifth vertical row it is shown again as 0.082 per cent of silicon. If two lines were drawn through these points they would represent the sloping sides of a segregation pyramid and would intersect at point 0.220 per cent or its apex. This effect can be explained in two ways: (1) that columnar crystals grow perpendicularly to the surfaces of the ingot mold regardless of the shape of the mold; and (2) by the effect that this type of solidification has upon the segregation of the ferrous-silicate inclusions relative to the ingot. This latter point is considered under a discussion of the mechanism of deoxidation.

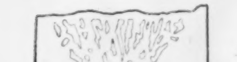
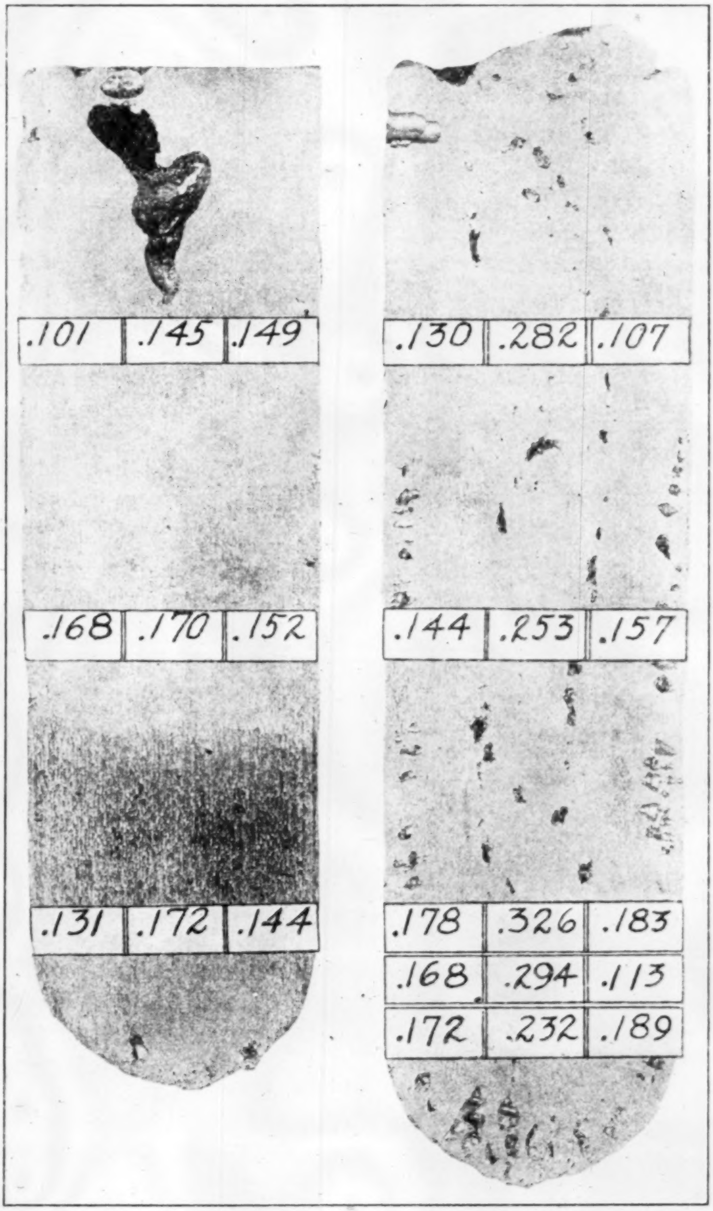


FIG. 3.—Showing Distribution of Silicon in 10½-ton Killed Ingot. Cast Analysis, 0.13 per cent of Silicon.

The Committee on the Heterogeneity of Steel Ingots for the British Iron and Steel Institute reported a similar segregation of silicon in a 10½-ton killed ingot, represented by Fig. 3. The sample analyzing 0.17 per cent of silicon represents the apex of the segregation pyramid in this ingot.


.101
.145
.149
.168
.170
.152
.131
.172
.144
.130
.282
.107
.144
.253
.157
.178
.326
.183
.168
.294
.113
.172
.232
.189

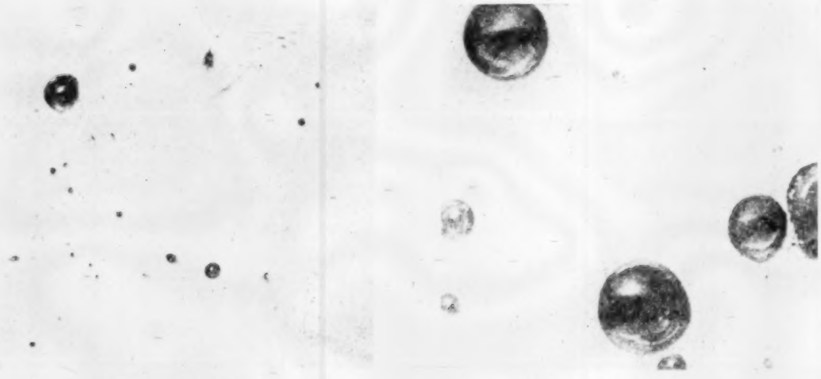
(a) Ingot No. 9 (Killed Steel). (b) Ingot No. 6 (Rimmed Steel).

FIG. 4.—Segregation of Silicate Inclusions in Rimmed and Killed Steels.

Segregation of Silicates:

Corresponding samples were cut from the opposite sides of these two split ingots, and the inclusions in these samples were extracted by the Dickenson method. The percentages of non-metallics extracted are represented in Fig. 4 at their respective positions on the two ingots.

The killed steel (ingot No. 9) shows some segregation of the non-metallic silica appearing higher in the vertical center and towards the bottom. Practically no FeO is present in these silica inclusions, and they are moved toward the vertical center line by at least two forces. The formation of columnar crystals perpendicular to the ingot walls tends to move any foreign silica particle toward the center, and surface tension effects would cause the particles to tend to stay with the liquid metal; the last liquid to solidify contains for these reasons the greatest amount of inclusions. This is illustrated by the point designated 0.172 (representing percentage of inclusions) and is near the apex of the segregation pyramid. That these silica particles increase in both size and numbers as they approach this vertical center is shown in Fig. 5(*a*) taken on the outside sample (represented by 0.144 per cent of inclusions) and in Fig. 5(*b*), which was taken of a center sample (containing 0.172 per cent of inclusions).



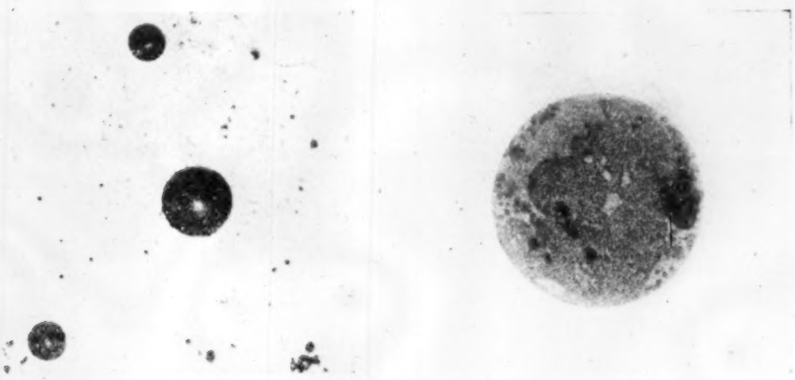
(*a*) Outside of Ingot. (*b*) Center of Ingot.

FIG. 5.—Photomicrographs of Inclusions in Ingot No. 9 (× 300).

Non-metallic segregation in a rimmed steel is represented by the samples of ingot No. 6, Fig. 4 (*b*). This segregation is very similar to that shown previously for silicon and is very high in the vertical center of the ingot and highest at 0.326 per cent, the apex of the segregation pyramid.

The same forces of columnar crystal formation and surface tension cause this central segregation as was the case in the killed steel (ingot No. 9) but the segregation here is accentuated by the rimming action in the mold. This segregation is also shown by Figs. 6(*a*) and (*b*). The photomicrograph shown in Fig. 6(*a*) was taken at the outside of the ingot No. 6 (at 0.183 per cent of inclusions) and that shown in Fig. 6 (*b*) at the center (at 0.326 per cent of inclusions). It is plainly shown by these two photomicrographs that the outside and center inclusions differ widely in size and appearance.

As the liquid steel is partly deoxidized, silica is formed and some excess of ferrous oxide is present. Then, when the steel is poured and columnar crystals start to form they will entrap some silica particles, but the majority will tend to stay with the liquid metal. These columnar crystals can contain only a small amount of FeO in solution, and due to their purity the liquid iron will gradually increase in FeO content. Thus the silica inclusions will combine with increasing amounts of ferrous oxide as the center of the ingot is approached. This mechanism continues until the last drop of metal solidifies. The greatest concentration of large high-iron oxide inclusions could be expected in that region.



(*a*) Outside of Ingot. (*b*) Center of Ingot.

FIG. 6.—Photomicrographs of Inclusions in Ingot No. 6 (× 300).

Findings of Howorth.—One of the first noteworthy investigations on the segregation of silicates in steel was carried out by Howorth. In tracing down a number of failures in gun steel he concluded that they were due to greenish slag particles which, upon extraction and analysis, proved to be iron-manganese silicates. He noted that in the majority of cases these slag particles were most numerous in forgings from the bottom sections of the ingots, and concluded that silicates formed on deoxidation must be highly segregated at that point.

Findings of Dickenson.—The most complete work to date on the distribution of silicates in steel ingots including tests on ingots up to 50 tons in weight has been carried out by Dickenson; Fig. 7 is a typical result. The maximum segregation in this ingot was found at the apex of the pyramid; similar segregation was found in the experimental work of the present authors. Dickenson also showed on a 36-in. octagonal ingot that there was an annular ring in the upper half of the ingot where the silicates were present in greater quantities than in the columnar wall on the outside or in the core within. This result was not obtained in the experimental work recorded herein, possibly due to the method of sampling (see Fig. 4). Such rings have been noted, however, in other experimental 20-lb. ingots deep-etched with hydrochloric acid.

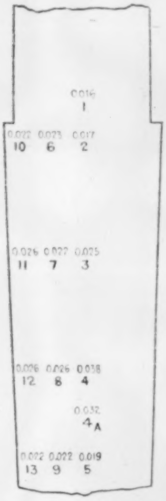


FIG. 7.—Silicate Results, According to Dickenson.

FORGING PROPERTIES OF STEELS CONTAINING VARIOUS TYPES OF IRON SILICATES

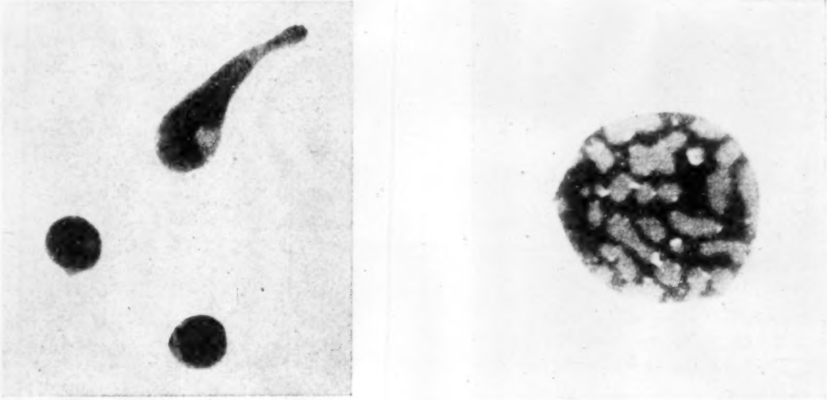
In order to obtain data on the effect of the various types of inclusions on the hot working of steel, and conversely of the hot working of steel on the inclusions, the experimental ingots were forged to 1-in. rounds.

TABLE II.—EFFECT OF TYPE OF SILICATE ON FORGING.

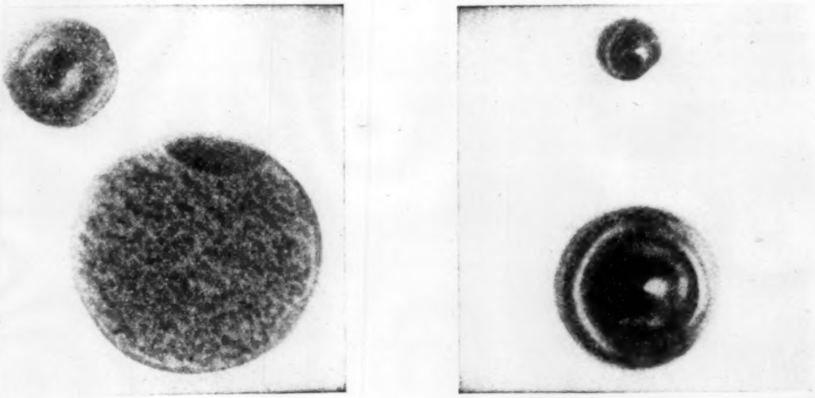
Ingot	Total Silicon, per cent	Type of Silicate	Forging Property
No. 1	0.004	See Figs. 8(*a*) and 9(*a*)	Good.
No. 4	0.021	See Figs. 8(*b*) and 9(*b*)	Fair.
No. 6	0.050	See Figs. 8(*c*) and 9(*c*)	Forged with difficulty.
No. 9	0.212	See Figs. 8(*d*) and 9(*d*)	Very poor.

It was observed that relatively good forging was effected in the presence of FeO and silicate inclusions high in FeO; but if the silicon was high enough to form the high-silica or glassy type of inclusion, increasingly higher forging temperatures were necessary—even then the steel cracked and would not weld upon further forging. This comparison may be found in Table II which compares the silicon present in four ingots with the type of silicate inclusion and finally with the forging properties.

The reasons for this increasing resistance to forging may be determined by a study of the effects of forging upon the inclusions themselves. This can be brought out by comparing the appearances of the various types of inclusions in cast and forged steels, as shown by Figs. 8 and 9.



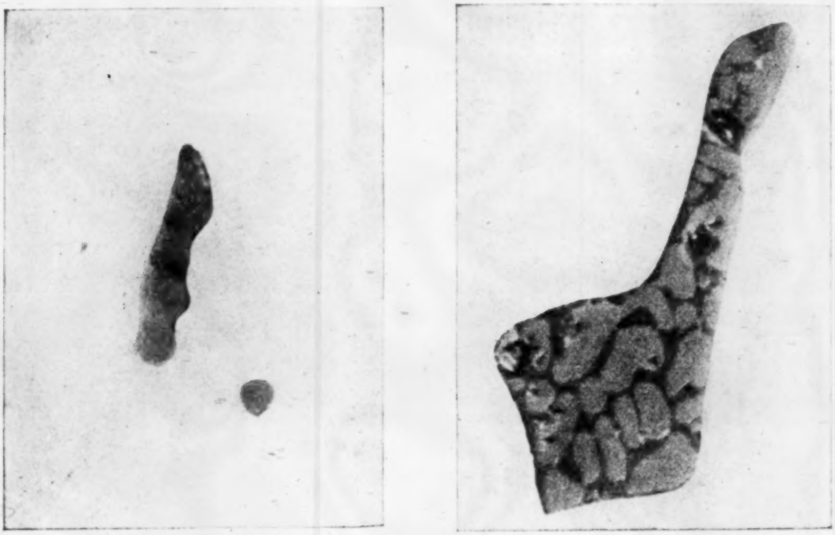
(*a*) Ingot No. 1 (× 1000). (*b*) Ingot No. 4 (× 500).



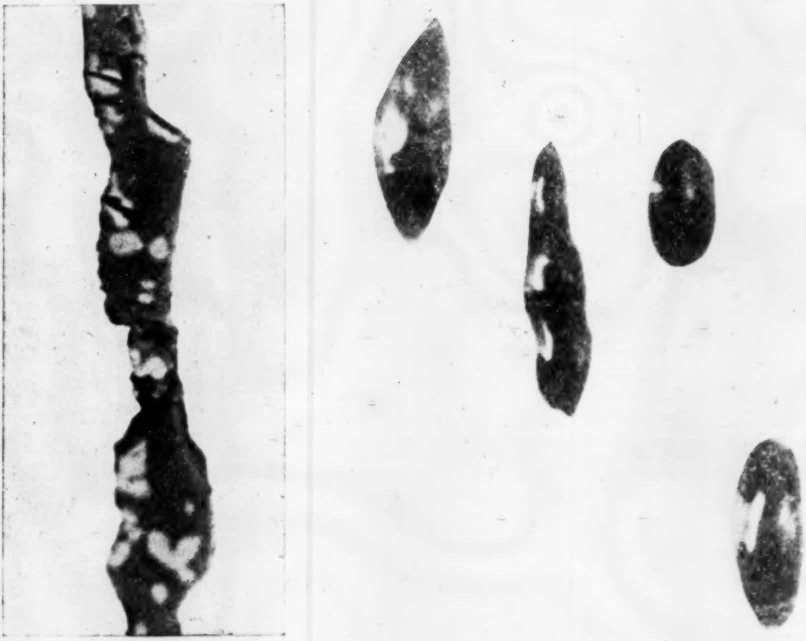
(*c*) Ingot No. 6 (× 500). (*d*) Ingot No. 9 (× 500).

FIG. 8.—Photomicrographs of Inclusions in Ingots Nos. 1, 4, 6 and 9.

By comparing the cast and forged FeO inclusions it was observed that only the larger particles were elongated, while the smaller inclu-



(a) Ingot No. 1 (× 1000). (b) Ingot No. 4 (× 500).



(c) Ingot No. 6 (× 500). (d) Ingot No. 9 (× 500).

FIG. 9.—Photomicrographs of Inclusions in Forged Ingots Nos. 1, 4, 6 and 9.

sions retained their spherical shape during forging, even though they were either pasty or liquid at the forging temperature. Figures 8(*a*) and 9(*a*) show this type of inclusion before and after forging. The high-FeO ferrous silicates in Figs. 8(*b*) and 9(*b*) show that inclusions of this type are considerably elongated by forging, due to their large size and liquid condition during forging.

The transition type of inclusion, about 65 per cent of SiO_2, would be more or less pasty at forging temperatures, and if present in appreciable quantity would form increased resistance to forging. Inclusions of this type are illustrated in Figs. 8(*c*) and 9(*c*).

High-silica inclusions cause the steel to become very resistant to forging because they are solid at forging temperatures. Some elongation is present, however, and may be seen by comparing Figs. 8(*d*) and 9(*d*).

The type of slag found in wrought iron is similar to that shown in Fig. 9(*b*). This inclusion is so liquid at forging or rolling temperatures that there is little tendency for the steel to be red-short. However, if the wrought-iron slag should become somewhat higher in silica, a tendency toward red-shortness would be predicted. In rimming steel the silicates are generally present as high-iron and high-manganese silicates, and these should cause no trouble during rolling. On the other hand, if any high-silica inclusions are present, such as would result from an incomplete cleaning of the steel if it had been re-boiled in the furnace with ferro-silicon, or if large particles of silica had been present in the bath due to poor scrap or poor quality hot metal, there would be a tendency toward cracking up of the ingot during rolling or toward seams in the finished product.

In killed steels, silicates are usually present as manganese silicates, and if the silica content of these silicates is high enough they will unquestionably cause red-shortness in rolling, particularly if the sulfur content of the metal is also high.

INCLUSIONS FORMED IN THE OPEN HEARTH ON THE ADDITION OF SILICON

In ordinary furnace operation, silicon is seldom if ever the only deoxidizer used. In most grades of killed or semi-killed steel the two common deoxidizers are silicon and manganese, and in many grades of steel three or more deoxidizers are common practice. Silicon, manganese and aluminum, or silicon, manganese and one other deoxidizer, such as titanium, zirconium, etc., are the usual combinations, and the effect of the deoxidation product of any one on the

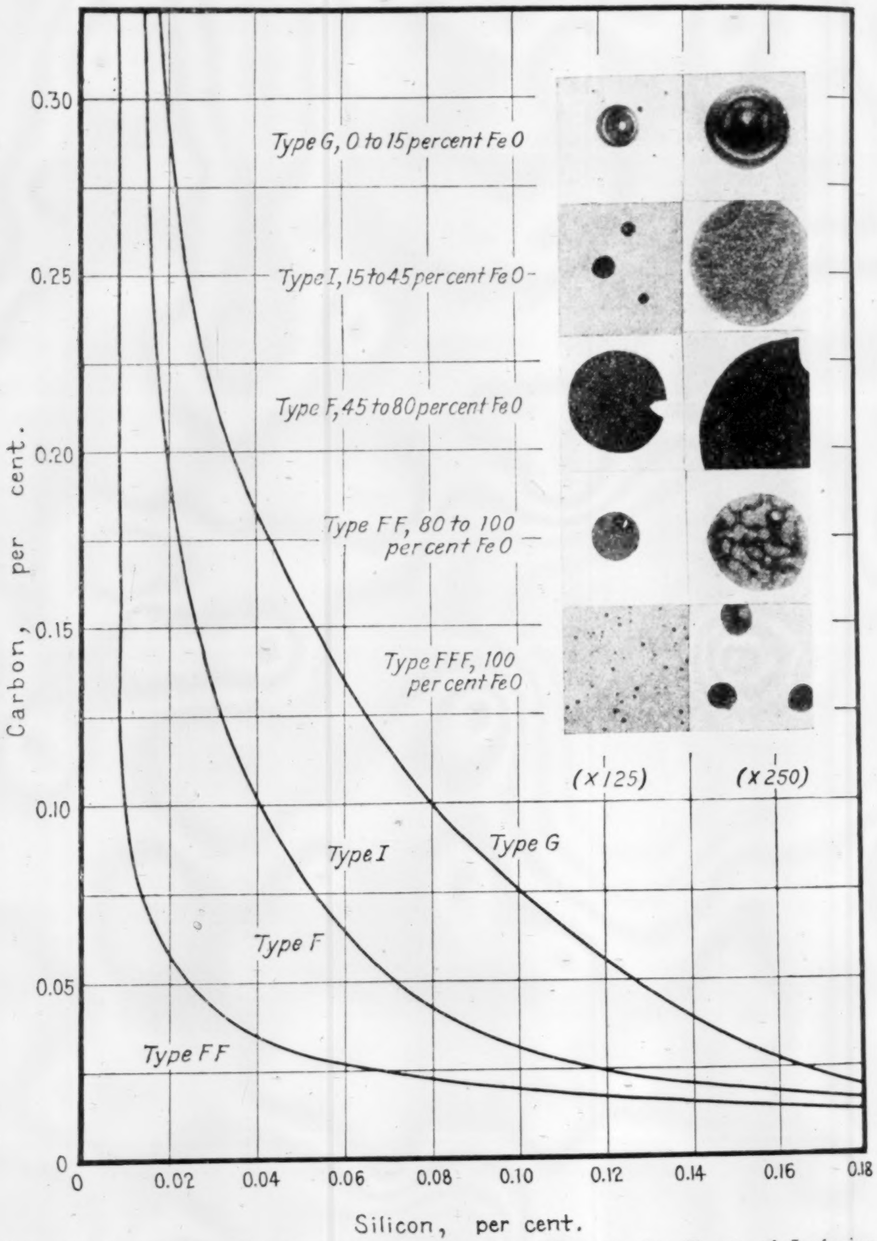


FIG. 10.—Comparison of Silicon and Carbon Contents with the Types of Inclusions Formed by Deoxidation of Ferro-Silicon.

The photomicrographs reproduced here were originally taken at magnifications of 250 and 500, respectively, but were reduced one-half in reproduction.

deoxidation products of the other two is difficult to predict without extended research.

There is one period of the open-hearth process, however, when silicon alone is used, and the results obtained at this point frequently determine the quality of the finished steel. For many grades of steel it is common practice to re-boil the heat with 10 to 15-per-cent silicon pig, or to kill the heat in the furnace in order to "hold" the carbon until an analysis of carbon may be obtained from the laboratory. When the heat is re-boiled, enough silicon is added to the bath to partly deoxidize the steel, and the heat is left in the furnace until the steel has taken up sufficient iron oxide to start the reaction between carbon and iron oxide. The final additions are added when the melter is satisfied that the steel and slag are in good shape to take these additions. When the heat is simply held with silicon until a carbon analysis is returned from the laboratory, the heat may be tapped dead if the carbon is at the desired point, re-boiled if the carbon is too high, or tapped and re-carburized with coal if the carbon is too low.

When silicon is added to the furnace the following reaction takes place:

$$Si + 2FeO \leftrightarrows SiO_2 + 2Fe$$

If an excess of FeO is present the silica fluxes with it, forming ferrous silicates whose composition varies with the amount of silica and iron oxide present and with the time allowed for fluxing. From a consideration of the equilibria between carbon and iron oxide, and silicon and iron oxide,[1] it is possible to predict the type of inclusion which will be formed on the addition of any given amount of silicon to a bath containing a given amount of carbon. Figure 10 shows this graphically. As the carbon content increases in the steel the iron oxide content decreases, and the type of silicate formed with a given amount of silicon will change accordingly. In the low-carbon ranges, 0.06 to 0.13 per cent of carbon, high and medium iron-oxide silicates will form, whereas in high-carbon steels the glassy silica particles will always be formed. It has been shown in the preceding discussion that the glassy type of inclusion is not fusible at rolling or forging temperatures, and if these particles persist in the steel until solidification takes place they are certain to have detrimental effects during rolling or forging if present in sufficient amounts. The higher iron types, on the other hand, are fusible enough to be rolled into streaks and probably have little if any effect on the rolling of the steel.

It is evident that a fusible particle is preferable to one which will not be deformed during working, as in the latter case strains are set up by the movement of the metal around the stationary particle. If silicon alone were used as a deoxidizer it would be impossible to avoid the glassy silica particle in killed steels. Fortunately, manganese oxide (MnO) acts in much the same manner as FeO in forming a relatively inoccuous silicate. This is unquestionably an important function of manganese in steel—that is, to form sufficient manganese oxide for fluxing the silica that results from the silicon addition. Manganese silicates are very similar to iron silicates, except that their melting points are somewhat higher and a high-manganese manganese silicate appears under the microscope to be similar to a high-iron iron silicate. Furthermore, since in steels killed with silicon alone the silicates contain from 5 to 15 per cent of iron oxide, the addition of manganese oxide forms a double silicate of iron and manganese which should be even less harmful than a manganese silicate. The exact composition of the manganese silicates formed on deoxidation depends upon the amounts of manganese and silicon formed, whether these are added separately or as an alloy, the carbon content at which the additions are made and the temperature.

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[1] Unpublished Bureau of Mines report, 1928.

At the present time, information is insufficient to predict what type of silicate will be formed under any conditions. It can be generally stated, of course, that the higher the ratio of silicon to manganese the more glassy the resultant silicate and the more harmful this will be to the steel. On the other hand, there is the decided possibility that if manganese alone were added in sufficient quantity, a large amount of MnO would be formed in the steel and this, having a melting point higher than the temperatures in the open hearth, would probably exert as harmful effects as silica, if it remained in the steel. The determination of the range of composition of inclusions which would be most rapidly eliminated from the steel and least harmful in its effects if present is a matter of long and careful research.

TESTS OF SPECIMENS CUT FROM DIFFERENT PORTIONS OF STRUCTURAL STEEL SHAPES

By M. O. Withey[1]

Synopsis

Data from 386 tension and compression tests of specimens cut from parts of shapes used in recent column tests made for the American Society of Civil Engineers are presented.

The tests show that the modulus of elasticity was nearly constant. The tensile strength was uniform for specimens taken from a given shape, but ranged for different shapes about 22 per cent. The average yield point for the different shapes exhibited a spread of 33 per cent both in tension and in compression. The yield point in tension and in compression varied with the location of the test specimen to some extent. For the same location, the yield point in tension was approximately equal to the yield point in compression. The weighted yield point in tension for a given shape was a good measure of the ultimate strength of that shape in compression. For the shapes tested, the average ratios of yield point to ultimate strength in tension varied between 0.59 and 0.64, with an individual minimum of 0.50. For the nine heats tested the ratio of yield point from mill tests to yield point from University of Wisconsin tests ranged from 0.88 to 1.16 and averaged 1.02.

Purpose of Tests

In connection with certain column tests made at the University of Wisconsin, in cooperation with the Committee on Steel Column Research of the American Society of Civil Engineers, 204 tension and 182 compression tests were made on test specimens cut from the material of which the columns were fabricated. The tests were made primarily to ascertain the yield point and modulus of elasticity of the metal used in the column tests. Because the mechanical work of rolling differs both with the kind of shape and for different parts of the same shape, it was deemed advisable to test a goodly number of specimens cut from different parts of the shapes used in order that the magnitude and importance of these effects might be determined. Since a considerable amount of valuable data were obtained in these auxiliary tests, it was thought desirable to present to this Society a more detailed account of them than will appear in the report of the column tests.

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[1] Professor of Mechanics, University of Wisconsin, Madison, Wis.

MATERIAL AND SPECIMENS

Mill tests indicated that the material furnished conformed to A.S.T.M. Standard Specifications for Structural Steel for Bridges (A 7 – 24).[1] The test specimens were cut from 8-in. 32-lb. H-columns, 10-in. 20-lb. channels, 10-in. 15-lb. channels, 12 by $\frac{1}{4}$-in. plates; also single lengths of a 10-in. 50-lb. H-column, a 10 by $\frac{5}{16}$-in. plate, and a 5 by $3\frac{1}{2}$-in. angle. Nine heats made at three plants are represented in the material tested.

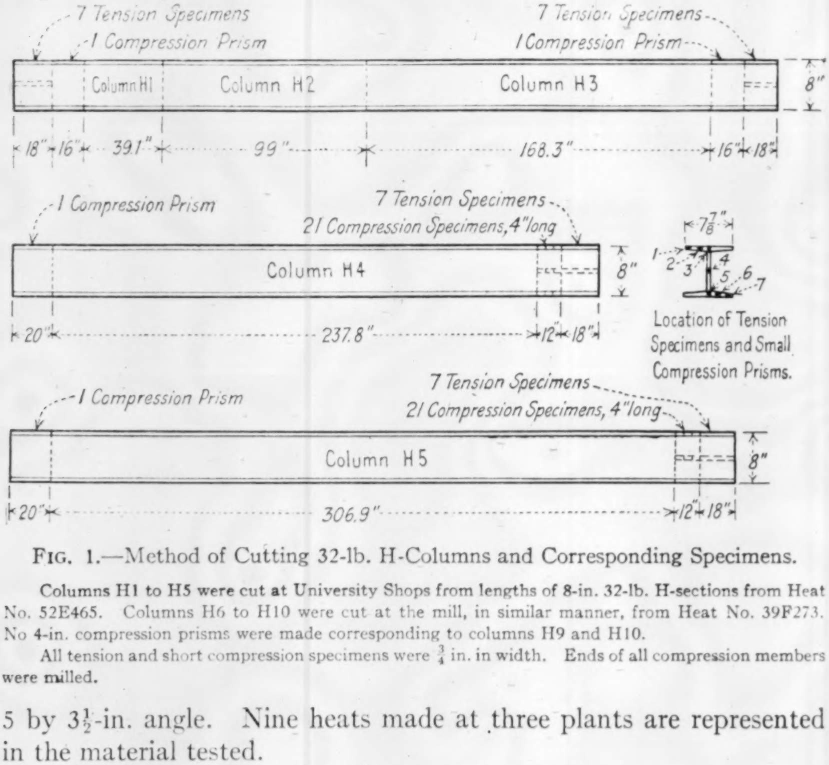
7 Tension Specimens
1 Compression Prism
Column H1
Column H2
Column H3
7 Tension Specimens
1 Compression Prism
8"
18"
16"
39.1"
99"
168.3"
16"
18"
1 Compression Prism
7 Tension Specimens
21 Compression Specimens, 4" long
Column H4
8"
20"
237.8"
12"
18"
$7\frac{7}{8}$"
1
2
3
4
5
6
7
Location of Tension Specimens and Small Compression Prisms.
1 Compression Prism
7 Tension Specimens
21 Compression Specimens, 4" long
Column H5
8"
20"
306.9"
12"
18"

FIG. 1.—Method of Cutting 32-lb. H-Columns and Corresponding Specimens.

Columns H1 to H5 were cut at University Shops from lengths of 8-in. 32-lb. H-sections from Heat No. 52E465. Columns H6 to H10 were cut at the mill, in similar manner, from Heat No. 39F273. No 4-in. compression prisms were made corresponding to columns H9 and H10.

All tension and short compression specimens were $\frac{3}{4}$ in. in width. Ends of all compression members were milled.

The sizes of specimens and the portions of the shapes from which they were obtained are shown in Figs. 1 and 2. The short lengths of shapes, from which the specimens were taken, were shipped to the laboratory with the columns and the specimens were then cut out by milling machine or shaper.

Cross-sectional dimensions of specimens were measured with a micrometer and were also calculated from the measured length and weight. Areas calculated from measurements averaged somewhat over one per cent larger than those calculated from weights.

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[1] 1927 Book of A.S.T.M. Standards, Part I, p. 63.

PROCEDURE IN TESTING

All tension tests were made in a two-screw 100,000-lb. Riehlé testing machine. In tension tests the speed of the movable head was 0.14 in. per minute at yield point and 0.65 in. per minute at maximum load. The compression tests were run at speeds of 0.068 in. per minute, or less, depending on the length of specimens.

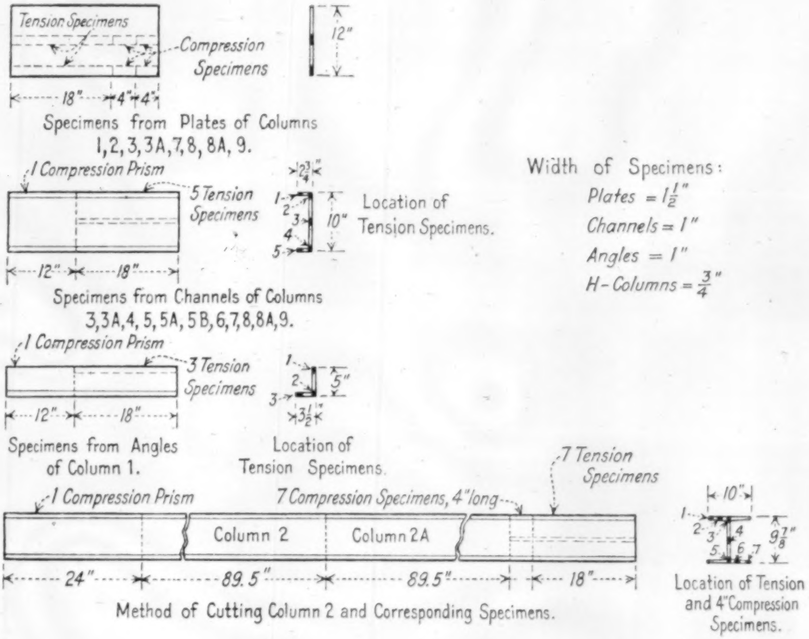
Tension Specimens
Compression Specimens
12"
18"
4"
4"
Specimens from Plates of Columns
1, 2, 3, 3A, 7, 8, 8A, 9.
1 Compression Prism
5 Tension Specimens
$2\frac{3}{4}$"
1
2
3
4
5
10"
Location of Tension Specimens.
12"
18"
Specimens from Channels of Columns
3, 3A, 4, 5, 5A, 5B, 6, 7, 8, 8A, 9.
Width of Specimens:
Plates = $1\frac{1}{2}$"
Channels = 1"
Angles = 1"
H-Columns = $\frac{3}{4}$"
1 Compression Prism
3 Tension Specimens
1
2
3
5"
$3\frac{1}{2}$"
12"
18"
Specimens from Angles of Column 1.
Location of Tension Specimens.
7 Tension Specimens
1 Compression Prism
7 Compression Specimens, 4" long
Column 2
Column 2A
24"
89.5"
89.5"
18"
Method of Cutting Column 2 and Corresponding Specimens.
10"
1
2
3
4
5
6
7
$9\frac{7}{8}$"
Location of Tension and 4" Compression Specimens.

FIG. 2.—Location of Specimens in Component Parts of Built-Up Columns.

In the compression tests of the whole prisms from the H-columns the tops and bottoms of specimens rested against roller bearings. In tests of half of these H-column prisms the axes of the bearings were normal to the webs; in the remainder they were parallel to the webs. The effect of this difference in procedure on the strength was negligible. In the other compression tests a spherical bearing surmounted the specimens. All whole prisms tested in compression were centered by trial.

A Moore indicating-dial extensometer reading to 0.0001 in. over an 8-in. gage length was used in all tension tests. A similar compressometer on a 2-in. gage length was used in the compression tests of small prisms. Berry strain gages with 8-in. gage lengths served to

TABLE I.—RANGES IN AVERAGE STRENGTH PROPERTIES OF TEST SPECIMENS.

Property Tested	Kind of Stress	Highest Average Stress, lb. per sq. in.	Shape Having Highest Stress	Lowest Average Stress, lb. per sq. in.	Shape Having Lowest Stress	Ratio, $\frac{\text{Highest} - \text{Lowest}}{\text{Specified Minimum}}$, per cent
1	2	3	4	5	6	7
Proportional Limit....	Tension.......	39 700	8-in., 32-lb. H-column	25 100	10-in., 20-lb. channel	49[a]
Yield Point...........	Tension.......	41 800	8-in., 32-lb. H-column	31 700	10-in., 20-lb. channel	33
Ultimate Strength.....	Tension.......	64 200	8-in., 32-lb. H-column	52 200	10-in., 20-lb. channel	22
Proportional Limit.....	Compression..	37 600	10 by $\frac{5}{16}$-in. plate	22 500	10-in., 15-lb. channel	50[a]
Yield Point...........	Compression..	40 900	8-in., 32-lb. H-column	30 900	10-in., 20-lb. channel	33[a]

[a] No minimum specified, based on 30,000 lb. per sq. in.

TABLE II.—PROPORTIONAL LIMITS IN TENSION OF SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL STEEL SHAPES.

Shape	Location of Specimen	Number of Specimens Tested	Proportional Limits				
			Minimum, lb. per sq. in.	Mean, lb. per sq. in.	Ratio to Whole	Average Variation, per cent	
						From Mean[b]	From Grand Mean[c]
1	2	3	4	5	6	7	8
8-in., 32-lb. H-Column........	Toe......	16	30 100	37 200	1.07	9.4	11.7
	Flange....	16	26 600	35 000	1.01	8.8	8.8
	Root.....	16	22 400	30 800	0.89	9.9	12.8
	Web......	8	28 800	37 400	1.08	7.5	11.8
	Whole....	8	......	34 800[a]	1.00	....	11.2
10-in., 20-lb. Channel.........	Toe......	35	14 700	28 400	1.03	10.8	11.4
	Root.....	36	18 500	27 300	0.99	11.0	10.6
	Web......	18	22 400	27 600	1.00	8.7	8.6
	Whole....	18	......	27 600[a]	1.00	....	10.5
10-in., 15-lb. Channel.........	Toe......	8	24 500	29 300	0.98	9.7	9.5
	Root.....	8	20 100	28 600	0.95	14.2	13.3
	Web......	4	28 500	31 500	1.06	8.0	8.6
	Whole....	4	......	29 900[a]	1.00	....	10.8
12 by $\frac{1}{4}$-in. Plate.............	Edge.....	11	22 200	32 900	0.98	9.3	8.3
	Center....	11	31 000	34 200	1.02	3.9	4.2
	Whole....	11	......	33 600[a]	1.00	....	6.3
5 by 3 by $\frac{5}{16}$-in. Angle.......	5-in. Toe .	2	28 000	29 700	0.89	5.7	12.6
	3-in. Toe .	2	38 500	39 200	1.17	2.7	15.2
	Root.....	2	32 200	33 100	0.99	1.7	2.6
	Whole....	2	......	33 500[a]	1.00	....	10.2

[a] Calculated by weighting specimens in proportion to volumes of adjoining metal.
[b] Mean for specimens located as indicated in column 2.
[c] Mean for all specimens of shape designated in column 1.

measure the deformations along the outer fibers of the whole prisms tested in compression.

Both in tension and compression tests, the load increments, decided upon prior to testing, were progressively applied and the corresponding deformations measured. In the compression tests of small prisms and in all tension tests the data were plotted as read. Yield points were determined both by drop of beam and by the increase in velocity of the pointer on the extensometer dial. The proportional limit was determined by noting the unit stress at which the stress-strain curve began to deviate from a straight line.

TABLE III.—YIELD POINTS IN TENSION OF SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL STEEL SHAPES.

Shape	Location of Specimen	Number of Specimens Tested	Yield Point				
			Minimum, lb. per sq. in.	Mean, lb. per sq. in.	Ratio to Whole	Average Variation, per cent	
						From Mean[b]	From Grand Mean[c]
1	2	3	4	5	6	7	8
8-in., 32-lb. H-Column........	Toe......	16	35 900	41 000	1.06	5.7	6.9
	Flange....	16	35 700	38 200	0.99	5.0	5.0
	Root.....	16	33 500	36 200	0.94	4.7	7.0
	Web......	8	38 200	40 500	1.04	3.9	5.1
	Whole....	8	......	38 800[a]	1.00	...	6.2
10-in., 20-lb. Channel.........	Toe......	35	28 200	33 900	1.04	4.4	5.7
	Root.....	36	27 600	31 400	0.97	3.6	4.8
	Web......	18	31 800	32 800	1.01	1.9	1.9
	Whole....	18	......	32 500[a]	1.00	...	4.6
10-in., 15-lb. Channel.........	Toe......	8	31 700	34 700	0.98	4.0	3.9
	Root.....	8	31 300	35 100	0.99	4.1	3.8
	Web......	4	35 300	37 400	1.06	5.6	5.9
	Whole....	4	......	36 000[a]	1.00	...	4.3
12 by $\frac{1}{4}$-in. Plate.............	Edge.....	11	34 600	36 900	1.01	3.0	3.1
	Center....	11	34 200	36 300	0.99	2.4	2.4
	Whole....	11	......	36 600[a]	1.00	...	2.7
5 by 3 by $\frac{5}{16}$-in. Angle.......	5-in. Toe .	2	36 300	36 400	0.95	0.3	5.5
	3-in. Toe .	2	40 300	41 200	1.08	2.1	6.8
	Root.....	2	37 700	38 000	0.99	0.7	1.4
	Whole....	2	......	38 300[a]	1.00	...	4.6

[a] Calculated by weighting specimens in proportion to volumes of adjoining metal.
[b] Mean for specimens located as indicated in column 2.
[c] Mean for all specimens of shape designated in column 1.

In order to determine the average yield point, or other property, for a given shape, the values for the test specimens cut from different parts of the shape were weighted in proportion to the volume of adjoining material. These average values for each shape were in turn averaged and are entered in the tables as the "mean" value opposite "whole." The mean values for whole specimens are used in most of the comparisons drawn.

RESULTS OF TESTS

Range in Quality of Steel Tested:

Tables I to IV summarize the strength data from these tests. Table I shows the range in the weighted average strength properties of whole specimens. It will be observed that the range in average proportional limits was greatest, being about 50 per cent. The average yield points were all within specification limits but the ultimate strength values for the 10-in. 20-lb. channels were all slightly below the minimum requirement. The range in strength properties of the individual specimens was considerably greater than the values recorded in Table I.

TABLE IV.—ULTIMATE STRENGTHS IN TENSION OF SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL STEEL SHAPES.

Shape	Location of Specimen	Number of Specimens Tested	Ultimate Strength			
			Minimum, lb. per sq. in.	Mean, lb. per sq. in.	Average Variation, per cent	
					From Mean[b]	From Grand Mean[c]
1	*2*	*3*	*4*	*5*	*6*	*7*
8-in., 32-lb. H-Column	Toe	16	58 200	61 700	3.5	3.1
	Flange	16	57 000	60 500	2.9	3.0
	Root	16	55 200	60 000	4.2	4.0
	Web	8	58 200	61 600	3.9	4.0
	Whole	8	......	60 800[a]	...	3.5
10-in., 20-lb. Channel	Toe	35	51 200	54 500	1.7	3.1
	Root	36	48 000	52 500	0.9	2.1
	Web	18	50 900	53 400	2.5	2.5
	Whole	18	......	53 300[a]	...	2.4
10-in., 15-lb. Channel	Toe	8	53 200	59 500	5.1	3.1
	Root	8	55 400	59 800	3.6	2.1
	Web	4	54 200	60 200	5.0	2.5
	Whole	4	......	59 900[a]	...	2.4
12 by $\frac{1}{4}$-in. Plate	Edge	11	56 800	59 100	2.5	2.5
	Center	11	55 300	59 000	2.2	2.2
	Whole	11	......	59 000[a]	...	2.3
5 by 3 by $\frac{5}{16}$-in. Angle	5-in. Toe	2	59 400	59 700	0.5	0.7
	3-in. Toe	2	60 100	60 400	0.5	0.5
	Root	2	60 000	60 300	0.5	0.5
	Whole	2	......	60 200[a]	...	0.6

[a] Calculated by weighting specimens in proportion to volumes of adjoining metal.
[b] Mean for specimens located as indicated in column 2.
[c] Mean of all specimens of shape designated in column 1.

Minimum and mean proportional limits and yield points for the specimens cut from different parts of the shapes are listed in Tables II and III.

Approximately 2 per cent of the individual specimens cut from the channels gave proportional limits in tension less than 20,000 lb.

per sq. in. and about 27 per cent had proportional limits under 25,000 lb. per sq. in. Considering the H-columns, which showed the highest strengths of the shapes tested, none of the individual specimens had proportional limits below 20,000 lb. per sq. in. and only 5 per cent had proportional limits below 25,000 lb. per sq. in. For yield point in tension none of the individual specimens from H-columns fell below 30,000 lb. per sq. in. whereas 6 per cent of the specimens from channels had yield points below that value. Most of the low values of proportional limit and yield point came from the roots and toes of the channels. Microscopic examinations of grip-end portions of three of the test specimens from channels exhibiting lowest values of proportional limit failed to disclose abnormalities in structure which would account for such low values. The structures of these specimens and of companion specimens from the same shapes were fine-grained and very uniform. Specimens from edges of plates also exhibited considerably more variation in their proportional limits than specimens taken from the centers of plates.

TABLE V.—PERCENTAGE ELONGATIONS IN 8 INCHES OF TENSION SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL STEEL SHAPES.

Shape	Location of Specimen	Number of Specimens Tested	Elongation in 8 in.				
			Minimum, per cent	Mean, per cent	Ratio to Whole	Average Variation, per cent	
						From Mean[b]	From Grand Mean[c]
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
8-in., 32-lb. H-Column	Toe	16	22.1	26.5	0.97	7.1	7.8
	Flange	16	25.0	28.7	1.05	5.2	7.4
	Root	16	22.4	26.2	0.95	8.1	7.7
	Web	8	26.0	28.2	1.03	4.6	5.3
	Whole	8	....	27.4[a]	1.00	....	7.3
10-in., 20-lb. Channel	Toe	33	13.2	19.1	0.68	15.6	25.6
	Root	34	20.8	28.6	1.01	8.7	13.6
	Web	16	21.5	31.6	1.12	7.6	25.8
	Whole	16	....	28.2[a]	1.00	....	20.8
10-in., 15-lb. Channel	Toe	8	16.9	20.6	0.89	13.9	14.8
	Root	8	13.0	22.0	0.95	14.4	13.9
	Web	4	22.0	26.1	1.13	8.7	17.8
	Whole	4	....	23.2[a]	1.00	....	15.0
12 by $\frac{1}{4}$-in. Plate	Edge	11	28.0	30.8	1.01	4.3	4.4
	Center	11	23.4	30.5	1.00	6.4	6.4
	Whole	11	....	30.6[a]	1.00	....	5.4
5 by 3 by $\frac{5}{16}$-in. Angle	5-in. Toe	2	30.2	30.3	1.10	0.3	9.8
	3-in. Toe	2	24.2	25.9	0.94	6.6	6.2
	Root	2	26.0	26.6	0.96	2.4	3.6
	Whole	2	....	27.6[a]	1.00	....	6.5

[a] Calculated by weighting specimens in proportion to volumes of adjoining metal.
[b] Mean for specimens located as indicated in column 2.
[c] Mean of all specimens of shape designated in column 1.

Table IV shows that for a given shape the variations in the ultimate strengths of the individual specimens were much less than the corresponding variations in yield point or proportional limit.

From Table V it appears that the percentage elongation of the metal in the toe specimens of the channels was markedly less than the percentage elongation of the web specimens and that there was much variation in the ductilities of individual specimens taken from different parts of the other shapes. A similar table for percentage reduction of area (not presented) shows that the variation in that property in the channel specimens was much less than was the variation in percentage elongation.

TABLE VI.—MODULI OF ELASTICITY IN TENSION OF SPECIMENS CUT FROM DIFFERENT PARTS OF STRUCTURAL STEEL SHAPES.

Shape	Location of Specimen	Number of Specimens Tested	Modulus of Elasticity					
			Maximum, lb. per sq. in.	Minimum, lb. per sq. in.	Average, lb. per sq. in.	Ratio to Whole	Average Variation, per cent	
							From Mean[b]	From Grand Mean[c]
1	2	3	4	5	6	7	8	9
8-in., 32-lb. H-Column.	Toe......	16	30 800 000	27 300 000	29 400 000	1.01	2.3	2.3
	Flange....	16	30 700 000	27 400 000	29 200 000	1.00	2.4	2.4
	Root.....	16	30 000 000	26 400 000	28 900 000	0.99	3.1	2.9
	Web......	8	30 700 000	29 200 000	29 800 000	1.02	1.3	2.1
	Whole....	8	....	....	29 200 000[a]	1.00	...	2.5
10-in., 20-lb. Channel..	Toe......	35	33 400 000	26 700 000	29 500 000	1.01	3.2	3.2
	Root.....	36	31 300 000	25 300 000	28 900 000	0.99	3.2	3.1
	Web......	18	31 200 000	26 100 000	29 200 000	1.00	3.2	3.2
	Whole....	18	....	....	29 200 000[a]	1.00	...	3.2
10-in., 15-lb. Channel..	Toe......	8	31 100 000	26 800 000	29 000 000	1.01	4.1	4.4
	Root.....	8	31 000 000	27 100 000	28 700 000	1.00	4.3	4.3
	Web......	4	28 900 000	27 300 000	28 400 000	0.99	2.0	2.0
	Whole....	4	....	....	28 700 000[a]	1.00	...	3.9
12 by $\frac{1}{4}$-in. Plate......	Edge.....	11	30 100 000	27 000 000	28 900 000	1.00	2.8	2.8
	Center....	11	30 800 000	25 200 000	28 700 000	1.00	3.9	3.8
	Whole....	11	....	....	28 800 000[a]	1.00	...	3.3
5 by 3 by $\frac{5}{16}$-in. Angle.	5-in. Toe .	2	29 500 000	27 600 000	28 600 000	0.99	3.5	3.4
	3-in. Toe .	2	29 100 000	29 000 000	29 100 000	1.00	0.2	0.2
	Root.....	2	29 100 000	29 500 000	29 300 000	1.01	0.7	1.0
	Whole....	2	....	....	29 000 000[a]	1.00	...	1.5

[a] Calculated by weighting specimens in proportion to volumes of adjoining metal.
[b] Mean for specimens located as indicated in column 2.
[c] Mean of all specimens of shape designated in column 1.

The moduli of elasticity in Table VI have the least variation of any property tested. The average value in tension was approximately 29,000,000 lb. per sq. in.

Effect of Location of Specimen on Mechanical Properties:

Study of column 6 in Tables II, III, V and VI, column 5 of Table IV, and columns 7 and 8 of Table VII, shows that the ultimate strengths and moduli of elasticity of the specimens tested were not materially affected by the location of the test specimen in the shape. Proportional limit, yield point, and percentage of elongation were influenced considerably by the location of the test specimen.

For the H-columns the specimens from the middle of the outstanding flanges gave values of proportional limit and yield point most nearly like the weighted averages for the whole shape.[1] Furthermore, these specimens varied less from the grand mean for the shape than specimens from the web. In elongation the web specimens were somewhat more representative of the whole shape than the flange specimens.

TABLE VII.—COMPARISON OF YIELD POINTS IN TENSION AND COMPRESSION OF SPECIMENS CUT FROM DIFFERENT PARTS OF THE SAME STRUCTURAL STEEL SHAPE.

Shape	Location of Specimen	Number of Specimens Tested		Mean Value, lb. per sq. in.		Ratio to Whole	
		Tension	Compression	Tension	Compression	Tension	Compression
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
8-in., 32-lb. H-Column	Toe	4	12	38 600	39 000	1.03	1.03
	Flange	4	12	36 600	36 200	0.97	0.95
	Root	4	12	36 100	36 600	0.96	0.96
	Web	2	6	40 200	41 400	1.07	1.09
	Whole	2	6	37 600	38 000	1.00	1.00
10-in., 50-lb. H-Column	Toe	2	2	37 000	36 600	1.02	1.03
	Flange	2	2	36 800	36 500	1.02	1.02
	Root	2	2	34 700	35 100	0.96	0.98
	Web	1	1	35 800	35 700	0.99	1.00
	Whole	1	1	36 100	35 700	1.00	1.00
10-in., 20-lb. Channel	Toe	15	15	34 000	32 600	1.05	1.03
	Root	16	16	32 000	32 000	0.98	1.01
	Web	8	8	32 200	30 900	0.99	0.98
	Whole	8	8	32 500	31 600	1.00	1.00
10-in., 15-lb. Channel	Toe	4	4	34 500	33 900	0.97	1.00
	Root	4	4	36 300	33 400	1.02	0.99
	Web	2	2	35 400	34 200	1.00	1.01
	Whole	2	2	35 500	33 800	1.00	1.00
12 by $\frac{1}{4}$-in. Plate	Edge	11	22	36 900	35 800	1.01	1.01
	Center	11	22	36 300	35 300	0.99	0.99
	Whole	11	22	36 600	35 600	1.00	1.00
10 by $\frac{5}{16}$-in. Plate	Edge	2	4	41 600	41 800	1.02	1.02
	Center	2	4	40 200	39 800	0.98	0.98
	Whole	2	4	40 900	40 800	1.00	1.00

For the channels the web specimens exhibited least variation and were the best representatives of the proportional limit and yield point but gave elongations considerably higher than the average elongations for whole channels. The elongations for the root specimens were more representative of the whole channels.

Specimens from the middle portions of the plates were less variable than specimens from edges and just as representative of both strength and ductility.

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[1] This agrees with the findings of the U. S. Bureau of Standards, see *Technologic Paper No. 328*.

Although only a few specimens from angles were tested, it is worthy of note that the specimens from the toes of the short legs had the highest proportional limits and yield points but gave the lowest ductility. The root specimens from these angles were the best representatives in these properties.

Comparison of Tensile and Compressive Properties:

Comparing the proportional limits of tension specimens with the proportional limits of compression specimens from similar portions of the same shape showed that there was a wide variation in these two values especially in the specimens from roots and toes of channels. For the H-column and plate specimens the mean proportional limit in tension was 1 to 8 per cent higher than the mean proportional limit in compression, but for the channels the mean proportional limit in tension was the greater by 20 per cent. Table VIII indicates similar discrepancies between the proportional limits of whole prisms in compression. Inasmuch as the variations in proportional limit of various parts of these shapes appeared to have little, if any, effect on the strength of the columns, whereas the yield point was a determinator of column strength, it is believed that the values of the latter are of far more importance.

TABLE VIII.—STRENGTH AND ELASTICITY OF WHOLE PRISMS CUT FROM STRUCTURAL STEEL SHAPES TESTED IN COMPRESSION.

Shape	Number of Specimens Tested	Proportional Limit, lb. per sq. in.			Ultimate Strength, lb. per sq. in.			Modulus of Elasticity, 1000 lb. per sq. in		
		Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
8-in., 32-lb. H-Column..	10	22 600	30 000	26 200 (34 800)[a]	35 300	41 800	38 500 (38 800)[a]	27 500	29 900	29 000 (29 200)[a]
10-in., 20-lb. Channel....	17	22 000	27 500	24 600 (27 600)[a]	30 400	35 400	32 200 (32 500)[a]	27 500	35 000	30 000 (29 200)[a]
10-in., 15-lb. Channel....	4	21 000	25 000	23 800 (29 900)[a]	32 400	35 100	34 100 (36 000)[a]	25 700	31 500	29 000 (28 700)[a]
5 by 3 by $\frac{5}{16}$-in. Angle....	2	32 000	36 000	34 000 (33 500)[a]	37 600	38 200	37 900 (38 800)[a]	29 400	30 700	30 000 (29 000)[a]

[a] Values in parentheses are calculated means from tension tests, see Tables II, III, and VI. Those in column 8 are mean yield points in tension.

From Table IX it appears that the average ratio of yield point in tension to yield point in compression for the nine heats tested varied between 0.99 and 1.05. The mean ratio for all heats was 1.01.

Table VII shows good agreement between the mean yield points in tension and compression for specimens from the same location in a shape. Good agreement also characterized most comparisons of individual pairs of tension and compression specimens. These data furnish additional evidence that the yield points of structural steel in tension and compression are approximately equal.

TABLE IX.—RELATIONSHIP BETWEEN YIELD POINTS IN TENSION AND COMPRESSION FOR SPECIMENS FROM THE SAME HEATS.

Shape	Heat Number	Number of Pieces Sampled	Number of Specimens Tested		Average Yield Point, lb. per sq. in.		Ratio, Tension to Compression
			Tension	Compression	Tension	Compression	
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
8-in., 32-lb. H-Column	52E465 and 39F273	6	56	10[a]	38 800	38 500	1.00
8-in., 32-lb. H-Column	52E465	2	14	42	37 600	38 000	0.99
10-in., 50-lb. H-Column	33E287	..	..	7	36 100	36 000	1.00
		1	7	1[a]	.. ...	36 500	0.99
10-in., 15-lb. Channel	13291	4	20	4[a]	36 000	34 100	1.05
		2	..	10	35 500[b]	33 800	1.05
10-in., 20-lb. Channel	21244	18	89	17[a]	32 500	32 200	1.01
		8	..	39	32 500[b]	31 600	1.03
12 by $\frac{1}{4}$-in. Plate	11010 and 904905	11	22	44	36 600	35 800	1.02
10 by $\frac{5}{16}$-in. Plate	925696	2	4	10	40 800	39 800	1.02
5 by $3\frac{1}{2}$ by $\frac{5}{16}$-in. Angle	48182	2	6	2[a]	38 300	37 900	1.01

[a] Full size prisms.

[b] Averages corresponding to those in column 7.

TABLE X.—RATIOS OF PROPORTIONAL LIMIT TO ULTIMATE STRENGTH AND OF YIELD POINT TO ULTIMATE STRENGTH IN TENSION FOR INDIVIDUAL SPECIMENS.

Shape	Number of Pieces	Number of Specimens	Ratio, Proportional Limit to Ultimate Strength		Ratio, Yield Point to Ultimate Strength	
			Minimum	Average	Minimum	Average
1	*2*	*3*	*4*	*5*	*6*	*7*
8-in., 32-lb. H-Column	8	56	0.39	0.57	0.59	0.64
10-in., 20-lb. Channel	18	89	0.27	0.52	0.52	0.61
10-in., 15-lb. Channel	4	20	0.32	0.49	0.50	0.59
12 by $\frac{1}{4}$-in. Plate	11	22	0.38	0.57	0.56	0.62
5 by 3 by $\frac{5}{16}$-in. Angle	2	6	0.47	0.57	0.60	0.64

Table VIII, column 8, indicates that the average of the tensile yield points was about 2 per cent higher than the ultimate compressive strength of the whole prisms.

Consideration of these data demonstrates that the weighted yield points in tension calculated for the whole shapes was a good index of the compressive strength of the corresponding whole prisms.

Considering whole prism tests in compression and weighted tensile averages for similar shapes, the values in column 11 of Table VIII show that the moduli of elasticity in tension and compression averaged approximately 29,000,000 lb. per sq. in. and 29,500,000 lb. per sq. in., respectively.

Ratios of Proportional Limit and of Yield Point to Ultimate Strength:

Table X shows the minimum and average values of ratios of proportional limit to ultimate strength and yield point to ultimate strength as found in the tension tests of individual specimens from these thin shapes. Minimum ratios for limit of proportionality of certain root and toe specimens of channels were quite low as would necessarily follow from the data in Table II. The minimum ratio of yield point to ultimate strength was 0.50, nearly twice the lowest ratio of proportional limit to ultimate strength. The average ratios of proportional limit to ultimate strength varied between 0.49 and 0.57 for the different shapes tested. Corresponding average ratios of yield point to ultimate strength varied between 0.59 and 0.64.

TABLE XI.—COMPARISON OF YIELD POINTS AND ULTIMATE STRENGTHS IN TENSION FROM MILL TESTS WITH RESULTS OF UNIVERSITY OF WISCONSIN TESTS.

Shape	Heat Number	Yield Point			Ultimate Strength		
		Results of University of Wisconsin Tests, lb per sq. in.	Results of Mill Tests, lb. per sq. in.	Ratio, Mill Tests to University of Wisconsin Tests	Results of University of Wisconsin Tests, lb. per sq. in.	Results of Mill Tests, lb. per sq. in	Ratio, Mill Tests to University of Wisconsin Tests
1	2	3	4	5	6	7	8
8-in., 32-lb. H-Column	52E465	37 600 39 600[a]	43 700	1.16 1.10[a]	61 100 61 700[a]	64 300	1.05 1.04[a]
8-in., 32-lb. H-Column	39F273	39 800 41 400[a]	42 200	1.06 1.02[a]	60 500 61 500[a]	60 900	1.01 0.99[a]
10-in., 50-lb. H-Column	33E287	36 100 35 800[a]	39 900	1.11 1.11[a]	60 000 59 700[a]	60 000	1.00 1.01[a]
10-in., 15-lb. Channel	13291	36 000 37 500[a]	36 200	1.01 0.97[a]	59 900 60 200[a]	61 500	1.03 1.02[a]
10-in., 20-lb. Channel	21244	32 500 32 800[a]	34 600	1.06 1.05[a]	53 300 53 400[a]	56 800	1.07 1.06[a]
12 by $\frac{1}{4}$-in. Plate	11010	35 000	34 000	0.97	59 600	56 200	0.94
12 by $\frac{1}{4}$-in. Plate	904905	36 900	34 700	0.94	58 800	57 400	0.98
10 by $\frac{5}{16}$-in. Plate	925695	40 800	36 100	0.88	63 400	60 100	0.95
5 by $3\frac{1}{2}$ by $\frac{5}{16}$-in. Angle	48182	38 300	37 100	0.97	60 200	60 500	1.00

[a] Based on tests of specimens cut from webs. Other values are based on weighted averages for whole shapes.

Mill Strength Tests Compared with Wisconsin Tests:

In Table XI the average of the weighted mean values of yield point and ultimate tensile strength for the whole shapes from each of the nine heats have been listed together with the corresponding mill test data. The ratio of mill yield point to University of Wisconsin yield point, in column 5, ranged between 0.88 and 1.16 and averaged 1.02. The ratio of mill ultimate strength to University of Wisconsin ultimate strength varied between 0.94 and 1.07 and averaged 1.00. If the University data had been based upon measured areas as were the mill data, all the above ratios would be raised about one per cent. If the first five heats in the table are considered, it will be found that the average ratio of mill yield point to University of Wisconsin yield point on the basis of tests of web specimens is somewhat lower (1.05) than if based on weighed averages (1.08).

CONCLUSIONS

1. The moduli of elasticity of these specimens of structural steel were approximately the same in tension and in compression. The modulus of elasticity was also practically independent of the location of the test specimen in the shape.

2. The ultimate strengths of specimens from a given type of shape were quite uniform regardless of location of test specimen in the shape. There was, however, a considerable spread (22 per cent) in the strengths of specimens taken from different shapes.

3. The proportional limits in these tests exhibited the greatest variability of the strength properties tested. This variability was most marked in the test specimens taken from toes and roots of channels. In most instances this variability was probably due to differences in mechanical work and to temperature in rolling. The variation of the proportional limit in different portions of a shape appeared to have little effect upon the compressive strength of whole prisms or the strength of columns built of the shapes.

4. There was a wide spread (33 per cent) in average yield points, both in tension and compression of material in different shapes. The yield point, both in tension and in compression, varied to some extent with the location of the test specimen in a given shape. The yield points in tension and compression were nearly the same for specimens taken from a given portion of a given shape.

5. The weighted yield point in tension proved a good measure of the strength of the whole prisms tested in compression.

6. For the thin shapes tested the average ratios of yield point to ultimate strength varied between 0.59 and 0.64. In no case did this ratio for an individual specimen fall below 0.50.

7. Basing judgment on weighted averages for whole shapes, the best representative specimens for the H-columns were obtained from the middle of the outstanding flanges. For the channels the web specimens were the best representatives in strength tests and the root specimens in elongation measurements. For the plates the specimens from the centers were somewhat less variable than the edge specimens and just as representative.

8. For the nine heats tested the ratio of mill yield point to University of Wisconsin yield point varied from 0.88 to 1.16 and averaged 1.02. The corresponding ratios for the ultimate strength are 0.94, 1.07 and 1.00.

Acknowledgments.—These tests have been made possible through the painstaking care and effective cooperation of the following student research assistants: Messrs. E. R. Summers, N. A. Christensen, J. C. Stowers, G. S. Paul and R. R. Schrader.

DISCUSSION

Mr. Thum.

MR. E. E. THUM.[1]—I am interested to see that Mr. Withey draws our attention to the differences between the proportional limit and the yield point. Engineers generally feel that these two figures are the same, and, given the figures on the yield point, base a design upon them, a matter which I should judge would be rather unsafe in the construction of a large structure such as a bridge or a ship hull, which would be figured as an elastic whole.

In Great Britain of recent years there has been a considerable quantity of so-called "D-steel," in high-strength ship plate and beams used not only for the construction of hulls for the Navy, but also for commercial vessels. This material contains 0.30 to 0.35 per cent of carbon, 1.1 to 1.4 per cent of manganese, 0.10 to 0.15 per cent of silicon and ordinarily is used in the unheat-treated condition. Temperature at the final rolling mill pass is closely controlled for proper normalizing effect. A temperature of 830 to 860° C. is the range aimed at. The specifications call for a minimum proportional limit of 38,000 lb. per sq. in., and the acceptance test is made by loading the specimen to 4500 lb. per sq. in., then gradually increasing to 38,000, then reducing to 4500 lb. per sq. in. If the permanent extension exceeds a tolerance of 0.0004 in. the material is rejected; if it is less than that tolerance, it is accepted.

Mr. Menefee.

MR. F. N. MENEFEE.[2]—Both the author of the paper and Mr. Thum make reference to the proportional limit. I am interested in this for I have never been able to find anyone who could give me a clear mental picture as to what proportional limit means; so I am asking that those who have been working on the subject help me a little. Am I right in assuming that there is a rather definite relationship between the proportional limit and Poisson's ratio? Some of my own unofficial tests would indicate that the proportional limit is dependent upon the reduction of cross-sectional area of the test specimen, which naturally increases the intensity of the load, decreases the area, causes the test specimen to elongate at a slightly more rapid rate, thereby throwing the ratio of stress to strain out of proportionality, but having nothing whatever to do with the elastic limit or yield point. In other words, we can go past the proportional limit any number of times and go back again to a lower load and in no

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[1] Associate Editor, *The Iron Age*, New York City.

[2] Professor of Engineering Mechanics, University of Michigan, Ann Arbor, Mich.

Mr. Menefee. way affect the elastic limit or yield point, or whatever you want to call it. I should like to get some other person's opinion on that and this seemed to me a good place to raise the question.

Mr. Thackray. MR. GEORGE E. THACKRAY.[1]—In reference to the remarks of the last speaker, it has struck me that the matter of the proportional limit is largely a function of the delicacy of the measuring apparatus. If the apparatus is capable of the smallest measurements the proportional limit will seem to be lower, and if the apparatus is only capable of coarser measurements the values obtained will be higher.

Mr. Tuckerman. MR. L. B. TUCKERMAN.[2]—About twenty years ago Edgar Marburg made the first series of tests that I have found in which the question of distribution of the properties of steel through these large rolled sections was examined. That was followed in the next year by an examination, not so complete, by H. F. Moore and E. L. Hancock, of certain rolled sections. Then in the years 1920 to 1925, the Bureau of Standards tried to find out why it was that consistent results were not obtained in its column tests. Looking back at those test results of Marburg the conclusion was reached that it might be desirable to find out what the distribution of strength properties was through these sections. In a later series of column tests between eleven hundred and twelve hundred test specimens were examined cut from different parts of the various sections.[3] The results found were similar to those which Mr. Withey has found, and it is of considerable interest that Mr. Withey's results confirm so well the type of results we obtained in those tests. An interesting point from the practical standpoint is the confirmation which Mr. Withey has found, that if you want a good representative specimen of a large section you can cut one out about half way down the flange and it will be as representative as any one specimen you can possibly get out of the material. This is not only true, as we found, for specimens showing the general behavior where the material shows the highest yield point and highest strength toward the outside and runs down at the root and perhaps climbs up a little in the middle of the web, but it is also true in cases where you are near the top of the ingot and segregation raises the values in the inside of the section. We tested, in the series, a number of columns which were cut off too near the top of the ingot, and the segregation at the top was sufficient that the yield point and tensile strength of the root of the flange and the web were markedly higher than they were out at the edge of the flange. Even in those cases the specimen

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[1] Special Engineer, Bethlehem Steel Co., Inc., Bethlehem, Pa.

[2] Engineer Physicist, U. S. Bureau of Standards, Chevy Chase, Md.

[3] L. B. Tuckerman and A. H. Stang, "Tests of Large Columns with H-shaped Sections," U. S. Bureau of Standards *Technologic Paper No. 328* (1926).

Mr. Tuckerman

cut half way down the flange gave a very good average value for the whole of the section.

In regard to the proportional limit, I have tried hard a number of times to find a proportional limit. I could always find a proportional limit at one value using an extensometer sensitive to 0.001 in. per inch. I could find another lower, using an extensometer sensitive to 0.00001 in. per inch and still lower using an extensometer sensitive to 0.000001 in. per inch; only when the limit of sensibility is stated does the proportional limit represent something which has to do with the elastic behavior of the material. Materials tested with extensometers of the same sensitivity will have different properties if they show different proportional limits. The significance of those differences is a matter of discussion.

I was interested in finding a modulus value reported as large as 33,100,000 lb. per sq. in. Every time I find a value as high as that on steel, I begin to wonder whether my testing procedure is quite as accurate as I thought it was, whether the testing fixtures have accurately averaged out the possible flexure of my specimen. When I have assured myself that the flexure is completely averaged out, I have never found anything as high as that.

In connection with yield point in compression and tension, Mr. Withey did not call attention to the difference in the speed of his testing machine in tension and compression, as perhaps affecting that ratio. If I remember rightly, he said that in compression the speed was 0.068 in. per minute and in tension 0.14 in. per minute; that is to say, it was about twice as fast in tension as it was in compression. Now, at speeds of about 0.01 in. or less per minute on ordinary structural steel, the yield point in compression usually ranges about 1.05 to 1.07 times the yield point in tension, and I think that the value approximating 1.00 or 1.01 which Mr. Withey has in this particular case depends upon the ratio of the speed of testing in the two cases, one in compression and the other in tension. It is interesting that where a comparative ratio of speed was compared in a previous report on yield point,[1] that ratio was 1.03; that is, over the whole range of speed of testing.

I noticed his method of determining areas consisted in measuring and then also making a check by weighing and computing from an assumed density. We have found that it is desirable, if one is going to the trouble of weighing, to go just a little further. If one will weigh the specimen in air and weigh it in water, which is a very simple

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[1] Report of the Research Committee on Yield Point of Structural Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 105 (1928).

Mr. Tuckerman. matter, he will free himself from any assumption whatever as to the density of the steel and can obtain area determinations with a reasonably cylindrical specimen which are much more accurate than can be obtained in any other way.

One other thing, the mill test results checked the results we found at the Bureau of Standards. The mill test specimen is a specimen cut from some sort of rolled shape from the same heat. It may be from the top of the ingot, from the bottom of the ingot, or it may be a relatively thin material or a relatively thick material, and a variation of approximately 20 or 30 per cent between the results of the mill test and the result of a coupon cut from a particular place in a particular specimen is just what one should expect under those circumstances.

In conclusion, I want to emphasize how desirable it is that people take the pains and trouble really to check up the properties of the material which they are testing in the form of structures. Many times a large amount of money has been spent in testing a structure, no provision being made for the small extra labor required for an adequate series of coupons necessary to determine the properties of the material in the structure. The test result of course represents the strength of that particular structure tested in that particular way. When, however, an attempt is made to interpret the test and draw from it conclusions applicable to other structures it is almost invariably found that the interpretation requires a knowledge of the properties of the material of which the structure was built. You can find in the literature of engineering testing case after case where the whole value of elaborate structural tests has been lost because no one knew the properties of the material in the structures.

A committee of the American Society of Civil Engineers, in providing for an adequate series of coupon tests of the materials, to accompany the tests of the columns, have given us results of permanent value. If someone, sometime, should make a column of somewhat different steel, he would still be justified in drawing conclusions from this series of tests since he can say, "I know the properties of my steel, and the tests of the committee have given me the relation between the properties of a steel and the strength of a column made from it."

Mr. Withey. MR. M. O. WITHEY[1] (*author's closure by letter*).—The limit of proportionality determinations in these tests were made from load-deformation graphs plotted as the readings were taken. The coordinate scales were such that load-deformation lines were inclined 30 to 60 deg. with the deformation axis and the lengths of the initial straight portions of these lines in most cases were between 4 and 8 in. **Mr. Withey.** With graphs of such size, or even larger, simple calculation shows that the elastic reduction in cross-sectional area, mentioned by Mr. Menefee, is too small to produce noteworthy curvature of the load-deformation line. I believe eccentricity in loading, non-homogeneity of structure and internal strain are the chief influences which lower the proportional limit. When the combined effect of these factors is small the limit of proportionality is near the yield point, when their combined effect is large it is much lower than the yield point.

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[1] Professor of Mechanics, University of Wisconsin, Madison, Wis.

In these tests the discrepancies in the proportional limits of different specimens were undoubtedly further augmented by differences in methods of rolling and in finishing temperatures used in fabricating the different shapes. It is of interest in this connection to note that the average ratios of proportional limit to yield point, from Tables II and III, for the H-columns, channels, plates, and angles were, respectively, 0.90, 0.84, 0.92 and 0.88. The relatively low ratios for the root specimens of the 15-lb. channels and the H-columns are also noteworthy.

Since sets were not determined the elastic limits cannot be exactly stated. However, I believe that in general the elastic limits, based on set determinations, would have been no higher than the corresponding proportional limits.

Publication of a reliable set of stress-strain curves for tests showing the relation of sensitivity of extensometer to proportional limit, such as Mr. Tuckerman mentioned, would make available the data for estimating the precision required in determining the proportional limit.

On account of the difficulty in adjusting the axis of the Moore extensometer to coincide with the axis of the specimen and on account of the presence of more or less eccentricity in the loading of the wedge-gripped thin specimens, it is probable that the recorded deformations differed from the corresponding axial deformations. Hence, as Mr. Tuckerman implied, it is likely that the high and low values of the modulus of elasticity were largely due to such errors in measurement.

In the compression tests on whole prisms, Table VIII, deformations were measured on four gage lines excepting in the tests of the angle specimens where there were three gage lines. Consequently, the effects of flexure were largely eliminated. The high and low values of the modulus of elasticity obtained in certain of these tests were probably caused by stress concentrations produced by inequalities in the ends and bearing surfaces.

TESTING OF MATERIALS AT ELEVATED TEMPERATURES[1]

BY P. G. MCVETTY[2]

SYNOPSIS

This paper discusses the general subject of materials testing as a basis for choice of materials and working stresses to be used in applications at elevated temperatures. The short-time and long-time tension tests are discussed in detail, including test methods, possibility of errors, and interpretation of results. The subject of accelerated tests is taken up with a discussion of the value of such tests as a basis for design.

The conclusion is reached that further work should be done to establish a reliable accelerated test or to correlate the short-time and long-time tension tests. In the present state of our knowledge, it appears that carefully conducted short-time and long-time tension tests with precision temperature control and strain measurement are the most reliable basis for the choice of materials and of safe working stresses for service at elevated temperatures.

INTRODUCTION

The growing demand for materials to be used at high temperatures is resulting in the development of a large number of alloys varying widely in properties and in cost. An accurate knowledge of these properties is the necessary basis for choice of the material best suited to the combination of conditions found in each particular application.

It is desirable that the choice be influenced by a comparison of allowable working stresses since this determines the amount of material required and the resulting cost. In addition to strength and ductility, it is necessary to consider resistance to scaling, corrosion, erosion, or any other condition which may affect the stability of the material in service. Long exposure to working conditions may result in the softening of heat-treated or cold-worked material. The determination of allowable working stress is complicated by the tendency of many materials to creep under relatively low stress when exposed to high temperatures. In many cases, this plastic deformation is of such small amount as to be negligible; in others, it may be a controlling factor. In such cases, allowable stress will depend upon allowable deformation within the expected life of the material.

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[1] The developments mentioned in this paper form a part of a general investigation being made by the author for the South Philadelphia Works of the Westinghouse Electric and Manufacturing Co.

[2] Mechanical Engineer, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

CHOICE OF MATERIAL FOR TESTS

In laying out any program of tests, the uniformity of the material must be a primary consideration. Complex alloys, suitable for high-temperature service, are often less homogeneous than plain carbon steels. Test specimens cut from the same bar sometimes show considerable difference in composition and physical properties. This makes it difficult to choose specimens representative of the material as a whole. The question of uniformity involves two separate and distinct problems. Unless these are considered separately in the selection of specimens, test results may be misleading.

Slight variations in commercial materials must be expected. This applies to heat treatment as well as to analysis and it would be desirable to have the tests cover the entire range of structural variations. Since the material may be particularly sensitive to slight changes in analysis or heat treatment, it is obvious that a study of uniformity even at one temperature requires a large number of tests. In addition, tests must cover the temperature range from normal to the maximum encountered in service. Furthermore, it is desirable to make more than one test under the same conditions on account of the effects of variations in testing procedure. From this it is evident that a complete study of the properties of one type of material at elevated temperatures would require a large number of tests. A comparison of the properties of many materials by this method would require a long time or an enormous amount of testing equipment. Many alloys are now available and more are being developed faster than suitable tests can be made. The most promising method of simplifying the problem is to divide it into two parts as follows:

1. A study of the probable variations in material and testing procedure at one temperature such as the highest temperature reached in service.

2. A study of the effects of several temperatures upon specimens selected and prepared with the greatest care to secure structural uniformity.

The first series of tests gives a measure of the differences which may exist between the properties of commercial material and those of the selected specimens. The second series gives a measure of the influence of temperature upon the type of material being studied. The selection of specimens for the first series is relatively simple, but the second lot requires the utmost care from beginning to end. The uniformity required of these specimens is much greater than could be expected under any commercial conditions.

It is impossible in the space available to discuss all the details involved in the selection of these specimens, but the following may be mentioned as being desirable:

1. Knowledge of history of material.
2. Selection of adjacent bars from the same rolling.
3. Microscopic examination of structure at ends of bars.
4. Numbering of specimens to show position in original bar.
5. Extreme care in heat treatment.
6. Hardness tests for uniformity.
7. Microscopic study of structure of coupon cut from each specimen.
8. Uniform speed, feed and depth of cut in the machining operation.
9. Test of finished specimens for straightness.
10. Magnetic tests[1] of finished specimens.

Choice of Tests

When Kirkaldy published his book in 1863,[2] the tension test was his basis for comparing materials. The gradual development since that time has led to a number of different kinds of tests at normal temperature, which attempt to duplicate the conditions found in service. These various static and dynamic tests have a definite place in the selection of materials and working stresses for normal temperature service, and it is expected that a similar development will be made in the high-temperature field. An example of such development is shown in the work of Tapsell and Clenshaw[3] at the British National Physical Laboratory.

At present, the greatest amount of work is being done on tension tests. This work includes the short-time tension test, the long-time tension or "creep" test, and attempts to correlate the results obtained from them. An excellent example of such an investigation is shown in the work of French, Cross and Peterson[4] at the Bureau of Standards.

A study of the literature on this subject brings out the wide divergence of opinion among investigators. Some maintain that the proportional limit as determined in the short-time tension test at any temperature is a definite property of the material and that creep cannot occur unless this stress is exceeded. Others doubt the existence of a proportional limit or a creep limit at elevated temperature. Some hold that a critical temperature depending on the amount of previous deformation must be exceeded before softening of cold-worked material begins. Others feel that hardening by heat treatment or by plastic strain produces an unstable condition which may return to equilibrium at any temperature if sufficient time be given. The author is now inclined to favor this latter view. Unfortunately, design cannot wait until these questions are settled. Development must proceed and the results obtained will depend greatly upon the closest cooperation between the metallurgist and the designer through the medium of suitable tests. Unless the designer is thoroughly familiar with the effects of variations which may exist in the materials and the effects of probable variations in testing procedure, he cannot logically determine the proper balance between safety and economy.

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[1] Thomas Spooner, "The Magnetic Analysis of High-Speed Steel," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 128 (1926).

[2] David Kirkaldy, "Experiments on Wrought Iron and Steel."

[3] H. J. Tapsell and W. J. Clenshaw, "Properties of Metals at High Temperatures," Special Reports No. 1 and No. 2 of the Department of Scientific and Industrial Research (1927).

[4] H. J. French, H. C. Cross, and A. A. Peterson, "Creep in Five Steels at Different Temperatures," U. S. Bureau of Standards, *Technologic Paper No. 362* (1928).

Short-Time Tension Tests

It will be impossible within the limits of this paper to discuss in detail all the elements entering into the development of equipment and methods for tests at elevated temperatures. Different investigators use different equipment and methods and the results are influenced by these variations. This is evident when an attempt is made to correlate the work of different laboratories as shown in the recent report of the Joint Research Committee of the A.S.M.E. and A.S.T.M. on Effect of Temperature on the Properties of Metals.[1] This report clearly shows the desirability of standardizing equipment and methods as soon as satisfactory standards can be established. Much has been written in describing present and past practice and this paper will attempt only to discuss the effects of some of the variables entering into the problem. It appears that a careful study of these variables must precede the question of standardization.

The importance of sensitivity of stress and strain measurement has been discussed elsewhere.[2] The matter of alignment of the specimen in the testing machine is also important. It has been shown[3] that bending increases the actual maximum stress in the outer fiber about 1.5 per cent for each 0.001 in. that the axis of a 0.505-in. diameter test specimen deviates from the axis of pull in the machine.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 139 (1927).

[2] P. G. McVetty and N. L. Mochel, "Tensile Properties of Stainless Iron and Other Alloys at Elevated Temperatures," *Transactions*, Am. Soc. Steel Treating, Vol. XI, p. 82 (1927).

[3] P. G. McVetty, Discussion of Paper by M. F. Sayre on "A New Type of Mirror Extensometer," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 665 (1926).

Even with carefully machined specimens having threaded ends fitted loosely to the grips and lubricated spherical seats between the grips and the heads of the machine, probable errors of 15 per cent in stress measurement are not uncommon and errors as great as 40 per cent have been observed. It should be noted that lack of homogeneity across the test specimen has an effect similar to that of eccentric loading. A study of these effects would require precise measurement of extension in at least three positions around the circumference of the specimen. In practice, measurements are usually limited to two positions, in which case the actual bending is probably greater than is indicated by the readings. The average of the two readings is an accurate measure of the average value of extension; hence, it is the measured value of proportional limit rather than that of modulus of elasticity which is affected by misalignment.

In connection with strain measurement, it is necessary to keep in mind that the extensometer measures the elongation of the gage length as a whole. The test specimen is usually an aggregate of crystals having random orientation and different strengths in different directions. Since stretch probably starts along a plane of slip in the crystal or crystals least favorably situated with respect to the principal stress, it is evident that the unit deformation will vary from point to point along the gage length. It is probable that some plastic deformation occurs at very low stresses and that the amount of this deformation gradually increases until it can be observed by the extensometer used. To detect the actual beginning of plastic deformation requires the ability to measure a change in length equal to the distance between atoms, which is of the order of fifteen billionths of an inch. This is about four-thousandths of the smallest change which the Martens extensometer will detect. As this is small relative to the simultaneous elastic deformation, the tendency is to neglect the early stages of plastic strain. It is evident, however, that the measured value of proportional limit, especially at elevated temperatures, must be interpreted in terms of the sensitivity of the extensometer used in its determination.

Elastic deformation as measured is also an average value, since the modulus of elasticity varies with the angles between the principal stress and the crystallographic planes of the individual crystals. Except in the study of single crystals, it is impossible to distinguish these differences and the average for the aggregate is used.

The stress-strain relations determine the so-called "elastic" properties of a material. It has been shown that the stress is an average value since metals are not strictly homogeneous and isotropic and since perfect alignment in the testing machine is seldom attained. Strain is also an average value since the unit deformation may be expected to vary along the gage length. The modulus of elasticity is necessarily an average since the modulus of the individual crystals is different in different directions. Properties based on the stress-strain curve must be interpreted with these variations in mind. It is not the intention of the author to discredit the short-time tension test, but it is important that test results be interpreted with a full knowledge of their limitations.

In addition to stress and strain measurement, tests at elevated temperatures require uniformity of temperature throughout the gage length of the specimen during the progress of the test. In a recent paper[1] Mr. Kirtland Marsh gives an excellent discussion of this phase of the problem. The importance of careful furnace design cannot be overestimated. The presence of temperature gradients within the gage length is accompanied by corresponding variations in the tensile properties being measured. Here again, the results indicate the average condition of the specimen. The probable errors in this case depend on the material and the test conditions.

The method of measuring the actual temperature of the test specimen varies among different investigators. It is difficult to find a method that is completely satisfactory in all respects and the choice usually requires a balance of advantages and disadvantages. Mr. Marsh shows good results for a thermocouple clamped on the shoulder at the end of the specimen. We have secured similar results with a shorter furnace by using an external spring to hold the couple in contact with the surface of the specimen at the center of the gage length.[2] Precision in this measurement is very difficult to obtain and temperature surveys under testing conditions are necessary.

Changes in temperature during the progress of a test become of great importance if refinement of strain measurement is attempted. The resulting errors depend primarily on the instantaneous coefficient of expansion and the modulus of elasticity of the material at the temperature of the test. The effect of temperature changes on the extensometer itself may also be important. The material used for the extensometer parts within the furnace is usually chosen for its strength at high temperature rather than for a low coefficient of expansion. The temperature along the comparison members changes suddenly in going through the end of the furnace from a temperature higher than that of the specimen to one lower than that of the grips. With steep temperature gradients and corresponding changes in coefficients of expansion, it is almost impossible to analyze accurately the action of the extensometer during small temperature changes. On this account, the use of a sensitive extensometer should be confined to tests in which the temperature is controlled with corresponding precision. For example, a Martens type extensometer will be noticeably affected by temperature changes of the order of 0.1° C.

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[1] Templin, Braglio and Marsh, "Mechanical Properties of Aluminum Casting Alloys at Elevated Temperatures," *Transactions*, Am. Soc. Mechanical Engrs., 1928.

[2] P. G. McVetty and N. L. Mochel, "Tensile Properties of Stainless Iron and Other Alloys at Elevated Temperatures," *Transactions*, Am. Soc. Steel Treating, Vol. XI, p. 100 (1927).

It may be of interest to consider some of the precautions we use to maintain constant temperature and the resulting temperature changes. Current is supplied to the furnace from a transformer which is tapped to give one-quarter volt steps of voltage from 0 to 110 volts and one-half volt steps from 110 to 220 volts. This allows a fairly close adjustment of furnace current. Finer adjustment is obtained from a resistance in series with the furnace. This resistance consists of two rheostats in parallel providing coarse and fine adjustment in addition to the transformer steps. Temperature readings are taken on a potentiometer which indicates changes of the order of one microvolt or approximately 0.02° C.

Slow heating of the specimen in the testing machine is desirable, especially for tests at the lower temperatures, and we usually allow about four hours for bringing the test up to temperature. During this time the furnace current is adjusted to nearly the correct value by following a temperature record or frequent temperature readings. From the time the furnace is up to temperature until the end of the test, one man does nothing else but watch the potentiometer and adjust the transformer and series resistance so as to keep the furnace temperature as nearly constant as possible. Close attention is very necessary to avoid large deviations from the desired temperature. During this period, drafts across the furnace from open doors or windows should be avoided. Since furnace temperature is influenced by changes in room temperature, sudden changes in weather conditions may have an appreciable effect on temperature control.

The time required to produce conditions satisfactory for the start of the test varies, but equilibrium can usually be established in about five hours from beginning of heating if sudden temperature changes are avoided. Hand control is continued until after the yield point is passed. This method requires the services of three men during the actual test, which takes about one hour. The setting up of the test takes about an hour and working up the results takes from two to three hours. By combining computations with the heating of the next test, the labor per test averages about ten man-hours.

We have found it better to make one test of this kind per day rather than to attempt more tests conducted with less care.

It is of interest to consider what temperature variations are actually found when tests are conducted as outlined above. Taking our tests of stainless steel and nickel for the Joint Committee together with those of two other alloy steels recently tested, we have a total of 42 tests at temperatures from 100 to 600° C. An analysis of these tests on the basis of total variations in temperature up to the proportional limit shows the following:

Variations, deg. Cent.	Number of Tests	Percentage of Tests
0 to 0.2	5	11.9
0.2 to 0.4	10	23.8
0.4 to 0.6	8	19.0
0.6 to 0.8	4	9.5
0.8 to 1.0	5	11.9
1.0 to 2.0	6	14.3
2.0 to 3.0	2	4.8
3.0 to 4.0	2	4.8
	42	100.0

More than 75 per cent of the tests had variations of 1° C. or less, while 90 per cent fell below 2° C. variation. In spite of all precautions, 10 per cent showed variations of from 2 to 4° C. The amount of variation appears to be independent of the testing temperature and the average for the entire 42 tests is 0.82° C. If tests are conducted with less attention to temperature control, it is reasonable to suppose that the variations will be greater than indicated above.

Let us now consider the effect of such variations upon the test results. At 400° C., the instantaneous coefficient of expansion of a low-carbon steel is 0.000016. The change in length of a 3-in. gage length due to a temperature change of 1° C. is 0.000048 in. This is easily detected by a Martens extensometer and a change of less than 1° C. might easily cause an incorrect location of the proportional limit value. This shows how difficult it is to obtain an accurate measure of the proportional limit at elevated temperatures.

A gradual change in temperature throughout the test changes the slope of the stress-strain curve, thus introducing an error in the determination of modulus of elasticity. Figure 1 shows for an average value of modulus of elasticity and coefficient of expansion, the errors in the measured modulus per degree of temperature variation. It is evident from this curve that modulus determinations become less reliable as the measured value of proportional limit decreases. For a proportional limit of 2000 lb. per sq. in. the probable error in modulus is 25 per cent for each degree change in temperature during its determination. This makes the accurate determination of modulus of elasticity at high temperatures very difficult. In view of the difficulty of holding temperatures within one degree, it is not surprising that wide variations in proportional limit and modulus values are often found.

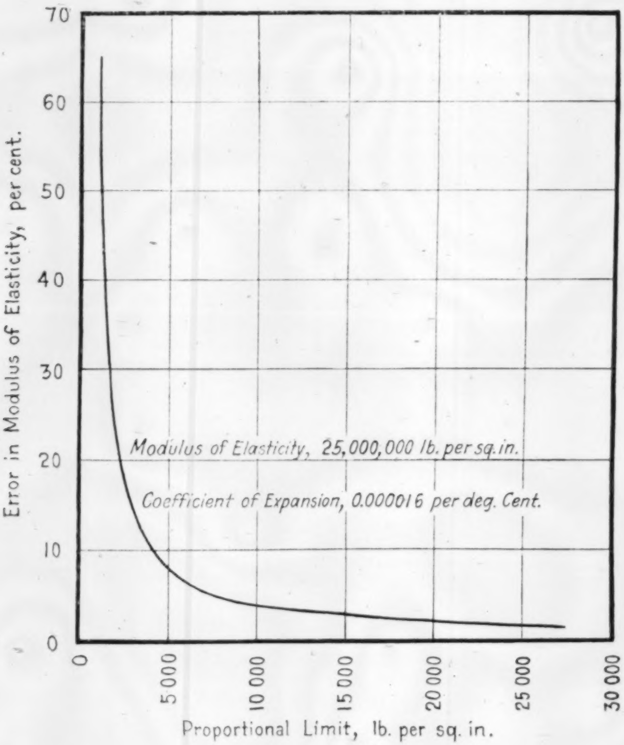

FIG. 1.—Effect of Temperature Change of One Degree Centigrade During Elastic Stage of Tension Test of Low-Carbon Steel at 400° C.

As the amount of deformation increases, the variations in length due to temperature changes become small in comparison with the total deformation. On this account the Johnson elastic limit and the yield point are less affected by temperature changes than is the proportional limit. The ultimate strength, elongation and reduction in area are usually only slightly affected by small changes in temperature during the test.

Testing speed is another variable which may have a decided influence on the test results. Errors due to variations in testing speed generally increase as the temperature rises and consistent results require similar rates of deformation in the several tests. The general effects of testing speed on strength and ductility are well known and need not be discussed here. The probable effects on proportional limit and modulus of elasticity appear to be less well known. Figure 2 shows the stress-strain curves for three tension tests of annealed medium-carbon steel at 400° C. The first represents the usual short-time test using a hydraulic testing machine and a Martens extensometer. The second refers to a test of the same material in which the testing time was five days. The third refers to a test which was extended over a period of five weeks. It is often assumed that a test can be unloaded without deviation from the elastic line

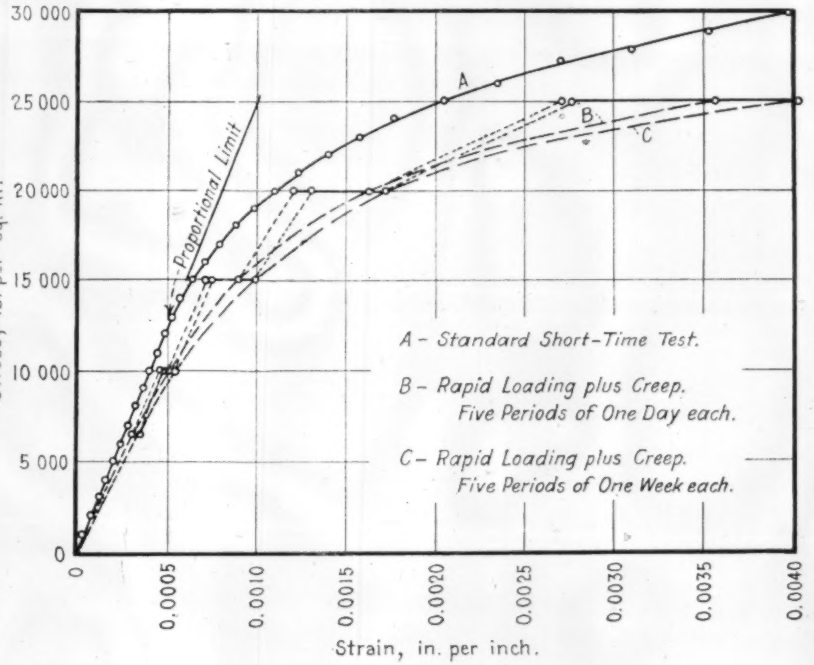

Fig. 2.—Tension Tests of Annealed Medium-Carbon Steel at 400° C. Using Martens Type Extensometer.

if the proportional limit is not exceeded. These curves indicate that the measured values of proportional limit and modulus of elasticity are lowered if the test is conducted very slowly. On this account, speed of testing must also be considered in comparing tension test results.

The variables which affect the determination of tensile properties of metal at elevated temperatures may be summarized as follows:

1. Method of selection of test specimens.
2. Uniformity of selected specimens.
3. Homogeneity of material in specimens.
4. Accuracy and sensitivity of stress measurement.
5. Accuracy and sensitivity of strain measurement.
6. Alignment of specimen in testing machine.
7. Speed of testing.
8. Uniformity of temperature throughout the test section.
9. Accurate measurement of temperature.
10. Close control of temperature during test.

The effect of temperature on the measured value of proportional limit is influenced by all of the above variables, and it is evident that test results are comparable only when extreme care is used in controlling test conditions.

Since the true proportional limit occurs at the stress which produces in one or more crystals a plastic strain equal to the distance between atoms, and since it is impossible to measure the stress on selected crystals in an aggregate, it is doubtful if true proportional limit can be measured commercially. The difference between the measured and the true value will in all cases depend upon the sensitivity of the extensometer and the precision of control of the variables involved.

The determination of effect of temperature upon true elastic limit is even more difficult because it involves loading and unloading with the possibility of error due to very small amounts of lost motion in the extensometer. On this account no attempt is made to measure elastic limit at high temperatures.

The effect of temperature on the modulus of elasticity may be determined roughly by ordinary tension tests but variations are to be expected unless exceptional care is taken in the control of testing speed and temperature.

The properties dependent upon the shape of the stress-strain curve during plastic deformation are mainly influenced by testing speed. An increase in speed tends to raise the Johnson limit and the yield point.[1]

The influence of very slow speeds of deformation has been shown in Fig. 2 and still slower rates will be discussed in connection with long-time tests. Ultimate strength, elongation and reduction of area are also influenced by speed of testing, the tendency of increased speed being to raise the ultimate strength and decrease the ductility.

Standardization of equipment and methods among different laboratories might be a decided disadvantage in the present state of our knowledge because of the possibility of restricting development. In any one laboratory, however, standardization of methods with the object of reducing variables to a minimum is highly desirable. When this is done, tensile properties have a definite meaning and the comparative value of many materials may be obtained by means of short-time tension tests. Results of such tests have been previously published.[2, 3]

LONG-TIME TENSION TESTS

In the long-time tension test, the stress and temperature are maintained approximately constant while extension is measured as a function of time. In a few cases, as in the work of Cournot and Sasagawa,[4] the stress is kept constant by suitable reduction in the load as the cross-section diminishes due to extension. When the study is confined to very small deformations, it is usually not necessary to go to this refinement.

Different laboratories have developed different equipment and methods and results depend to a certain extent upon the testing procedure. A comparison of results obtained under different conditions is given in the recently published work of Messrs. French, Cross, and Peterson.[5] They now confirm a statement previously made by the author in conjunction with Messrs. Lynch and Mochel[6] in finding an approximate agreement between the proportional limit as determined in the short-time tension test and the stress which can be sustained for long periods with small amounts of deformation. The importance of this conclusion is evident because it allows comparison of materials at elevated temperatures by means of the short-time tension test.

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[1] The definition of the term "Johnson limit" is well known but that of "yield point" requires some explanation. In tests at elevated temperatures, it is necessary to determine the yield point arbitrarily from the stress-strain curve. The practice of taking for this value the stress which produces a total deformation of 0.5 per cent introduces variable combinations of elastic and plastic strain. We have found it preferable to use a definition based on 0.2 per cent plastic strain or permanent set.

[2] P. G. McVetty and N. L. Mochel, "Tensile Properties of Stainless Iron and Other Alloys at Elevated Temperatures," *Transactions*, Am. Soc. Steel Treating, Vol. XI, p. 82 (1927).

[3] T. D. Lynch, N. L. Mochel and P. G. McVetty, "The Tensile Properties of Metals at High Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 5 (1925).

[4] J. Cournot and K. Sasagawa, "Contribution à l'Etude de la Viscosité des Alliages à Température Elévée," *Comptes Rendus*, 181, p. 661 (1925).

[5] H. J. French, H. C. Cross and A. A. Peterson, "Creep in Five Steels at Different Temperatures," U. S. Bureau of Standards, *Technologic Paper No. 362* (1928).

[6] T. D. Lynch, N. L. Mochel, and P. G. McVetty, "The Tensile Properties of Metals at High Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 18 (1925).

For the determination of safe working stresses, the proportional limit is not entirely reliable. If the allowable deformation is limited, it is safe to say that the allowable stress will not be greater than the proportional limit and this stress may be taken as a starting point in making long-time tests. Such tests are best made on the basis of some fixed limit of allowable deformation within the life of the material. The curve of elongation as a function of time generally shows a rapid stretch at first, followed by stretch at an approximately constant rate.[1, 2] It is usually possible to estimate whether the deformation will exceed the maximum allowable amount within the expected life of the material. If this appears to be the case, it is necessary to set up another test at a lower stress. This procedure must be repeated until a satisfactory stress is obtained. Unless the specimens used for these tests are very uniform, the results are difficult to interpret. On this account, the most complete information about the material and all possible preliminary tests for uniformity are desirable as a basis for selection of the specimens used.

It is sometimes difficult to determine where the initial flow stops and the secondary flow begins. This is especially true when the deformation is very small.[2] This makes the tests long and special care is necessary in temperature control and measurement of extension. These subjects require special consideration.

Let us assume that the maximum allowable plastic deformation is fixed at 0.001 in. per in. or 0.1 per cent during an expected life of twenty years. Of this amount 20 per cent may occur in the first 1000 hours of the test, leaving a total of only 0.0008 in. per inch to be detected if the test were continued for the remainder of the twenty years. At a constant rate of deformation, this represents a creep of 0.00004 in. per inch per year. Considering that a change in temperature of 2.5° C. would account for a change in length of this amount, it becomes evident that extreme sensitivity of strain measurement and temperature control are necessary in these tests if design is based on a very small deformation within a long span of life. It is understood, of course, that many applications allow greater deformations

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[1] T. D. Lynch, N. L. Mochel and P. G. McVetty, "The Tensile Properties of Metals at High Temperature," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 18 (1925).

[2] P. G. McVetty, Discussion of paper by H. J. French on "Methods of Test in Relation to Flow in Steels at Various Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 25 (1926).

and expect shorter life from the material. In such cases, the test requirements are simplified. For example, a plastic deformation of 1 per cent in a year might be allowed. This is about $\frac{1}{8}$ in. to the foot. Even neglecting the initial deformation, it would be necessary to measure with certainty a rate of stretch of 0.0002 in. per inch per week.

Whether the allowable deformation is large or small, it is evident that the time required to measure rate of deformation will be reduced as the sensitivity of strain measurement and temperature control is increased.

In this type of test, temperature control involves two different degrees of refinement. Prior to and during the time in which readings of length are taken, we find it desirable to use hand control as in the short-time tests. This is done to bring the test to equilibrium at the desired temperature. Enough readings are taken to ensure a good average value. At all other times, automatic temperature control of the potentiometer type is used. The main requirements of the latter are reliability and a continuous record of actual temperatures. Without such a record, temporary overheating may occur with a resulting change in length which may be misinterpreted.

We find it desirable to use a separate transformer, with the close voltage regulation previously described, to supply each furnace. A series resistance, which is periodically short-circuited by the automatic control, is adjusted to give the minimum fluctuations in furnace temperature. The value of this resistance depends on variations in line voltage and in room temperature. If the resistance is made too small, such variations may prevent accurate control. It is obvious that automatic control of line voltage and room temperature are logical steps toward improvement in the control of furnace temperature. Under our conditions, we are able to hold furnace temperatures at 400° C. within a total variation of 5° C. over long periods of time. In the present state of our knowledge, it is doubtful if changes of this amount have a serious effect on test results. It is known, however, that rate of creep is sensitive to temperature changes and better temperature control is expected to be a development of the near future.

Dependability of equipment is essential to avoid interference with tests which may run for several months. Our furnaces are wound with $\frac{1}{8}$ by $\frac{1}{32}$-in. nichrome ribbon and we have never had a furnace burn out in service. All of the electrical equipment is so arranged that any part may be quickly replaced in case of failure. This applies to transformers, automatic controls, relays, and series resistances. To avoid trouble, due to failure of power supply, two independent sources of power are available with an automatic change-over switch to put the entire equipment on the second source if the first fails. We feel that these precautions have fully justified their cost in avoiding loss of tests in which a large amount of time and labor had been expended.

INTERPRETATION OF TEST RESULTS

From the discussion of variables which affect test data, it is evident that the interpretation and comparison of test results require an accurate knowledge of the variables involved and their probable influence. Comparison of results from different laboratories can be made only after a careful analysis of the conditions under which tests are made. Both the short-time and the long-time tension tests have a definite field of usefulness: the former for comparing materials and the latter for determining safe working stresses. Attempts to correlate the results of the two tests are valuable but it is doubtful if the short-time test can replace properly conducted long-time tests.

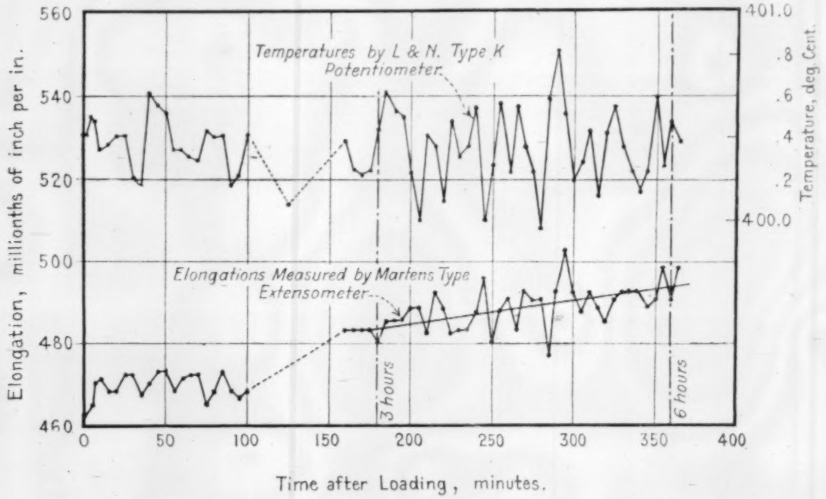

FIG. 3.—Creep Test of Annealed Medium Carbon Steel at 400° C. and 10,000 lb. per sq. in.

ACCELERATED TESTS

The need for rapid determination of allowable stresses has led to a careful consideration of accelerated tests. The recent work of Pomp and Dahmen[1] is based on a study of the rate of creep at constant stress and temperature between the third and sixth hour after the application of load. We have made a number of tests using the method of Pomp and Dahmen and we find that on account of temperature fluctuations, it is inadequate to measure the length at the end of the third and sixth hours as they suggest. Figure 3 shows readings of length and temperature during the first six hours after loading. Although the temperature variations are within one degree, the corresponding length changes interfere seriously with the determination of rate of creep. It becomes necessary to determine the rate for a given stress and temperature from the average curve through the observed points.

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[1] A. Pomp and A. Dahmen, "Entwicklung eines abgekürzten Prüfverfahrens zur Ermittlung der Dauerstandfestigkeit von Stahl bei erhöhten Temperaturen," *Mitteilungen* aus dem Kaiser-Wilhelm-Institut für Eisenforschung zu Düsseldorf, Vol. IX, p. 33 (1927).

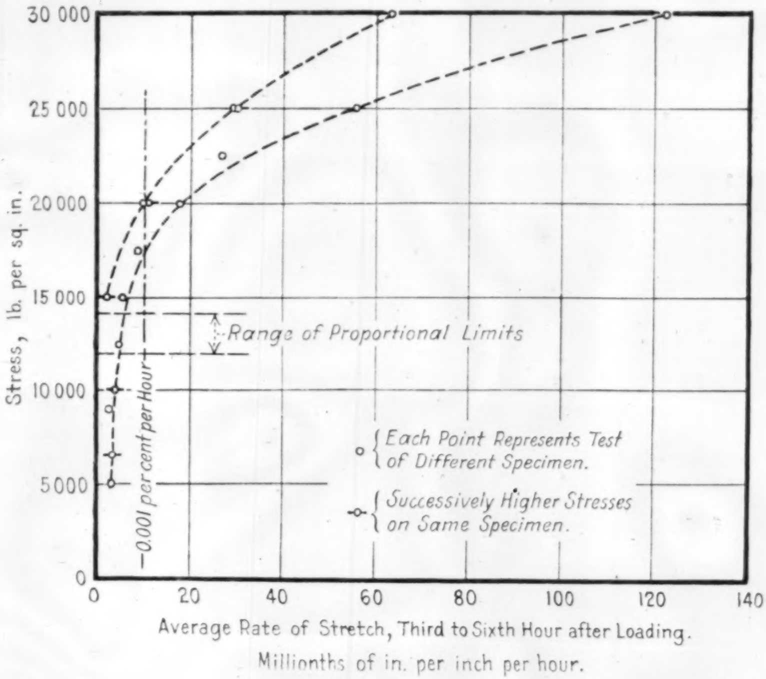

Fig. 4.—Comparison of Creep Limits by Pomp and Dahmen Method with Proportional Limits from Short-Time Tension Tests of Annealed Medium-Carbon Steel at 400° C.

Figure 4 shows the results of a number of such tests of medium-carbon steel at 400° C., in terms of the relation between stress and creep rate between the third and sixth hours. Pomp and Dahmen try to choose a stress giving a rate of 0.001 per cent per hour, by drawing a straight line between two points located on either side of a line representing this arbitrarily chosen rate of creep. It is evident from these data that the stress at which these two lines intersect will depend upon the particular pairs of points chosen and also upon the accuracy of measurement of creep rates in the original tests. An important objection to this method is the fact that the period studied is generally within the initial stage of creep and it is very doubtful if a safe prediction of the subsequent action of the material may be made from the data obtained. The work of French, Cross and Peterson, previously mentioned,[1] has a much more logical basis because a definite distinction is made between flow in the first and second stages.

Figure 4 shows also the range of variation in the measured values of proportional limit for this material at 400° C. Our experience with long-time tests indicates that these values are a better criterion of safe working stress than are the higher values obtained by the Pomp and Dahmen method.

It is not impossible that a satisfactory test quite different from the tension test may be developed. Even in a tension test, plastic deformation is in shear at an angle of 45 deg. with the principal stress. On this account, torsion tests are promising. For such tests, reference is made to the work of Tapsell and Clenshaw[1] and Pierre Henry.[2]

Since deformation probably results from slip within the crystals rather than between the crystals of an aggregate, it may be necessary to go to fundamental tests of single crystals as the basis for formulating a suitable accelerated test. The author now has such an investigation in process.

CONCLUSIONS

In this paper an attempt is made to show the effects of some of the variables encountered in making tests at elevated temperatures. The interpretation of test data is considered in reference to the influence of these variables and the resulting possibility of errors. In the present state of our knowledge, it appears that carefully conducted short-time and long-time tension tests with precision temperature control and strain measurement are the most reliable basis for the choice of materials and of safe working stresses for applications at elevated temperatures. Development of design, with proper regard for safety and economy, will require the closest cooperation between the designer and the metallurgist through the medium of suitable tests. A thorough understanding of the limitations of materials and of tests will help to put such design upon a sound basis.

Acknowledgment.—The author wishes to acknowledge the cooperation of Mr. N. L. Mochel and Mr. J. M. Lessells in the development of these test methods.

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[1] *Loc. cit.*

[2] Pierre Henry, "Recherches Experimentales sur les Vitesses de Déformation des Métaux aux Hautes Températures," *Revue de Metallurgie*, August 8, 1927, p. 421.

DISCUSSION

Mr. Cross.

MR. H. C. CROSS[1] (*presented in written form*).—The paper by Mr. McVetty is one which will be of value to those contemplating tests of metals at elevated temperatures. Conclusions drawn from so-called short-time or long-time tests may be misleading unless made with a full understanding of the many variables which enter into and influence such tests.

The author mentions a number of items relating to the selection of materials for high temperature tests and recommends hardness tests, microscopic examination and magnetic tests of finished specimens to guarantee uniformity. If Mr. McVetty has included such surveys in his work, it should be interesting and helpful to have him give an idea as to what extent such surveys should be carried out and what degree of uniformity is required for consistent results in the high-temperature tests.

A statement is included in the paper that the author inclines to the view that "hardening by heat treatment or by plastic strain produces an unstable condition which may return to equilibrium at any temperature if sufficient time be given." This is a very broad statement and can be interpreted to refer to steels which have been tempered at high temperatures subsequent to hardening and which are intended for use at temperatures well below the tempering temperatures. From a theoretical standpoint this may be so, but is this very broad statement justified from the standpoint of practical applications in all cases? No doubt, Mr. McVetty has some evidence obtained from his work to substantiate such a view, and it would be interesting to have the details and further comments as to just what is meant by the statement quoted.

In the interpretation of long-time test data, the chemical stability and resistance to oxidation and corrosion of the material must also be taken into consideration. Decarburization, grain growth, and scaling due to oxidation and corrosion may materially affect the results obtained and especially during tests run for long periods. Excessive scaling will materially reduce the cross-section and if tested for a long period of time will cause creep, whereas no creep would be noted for the same stress applied for only a short period of time.

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[1] Junior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

Mr. McVetty. MR. P. G. MCVETTY[1] (*author's closure by letter*).—Referring to Mr. Cross' question relating to the extent and reliability of our uniformity tests, I regret that I cannot give definite rules for such surveys because we have not yet developed a procedure which is entirely satisfactory. Our experience indicates that test specimens show more or less variation in properties even after great care has geen used in their selection and heat treatment. In spite of the usual assumption that the material being tested is homogeneous and isotropic, it is a well-known fact that this desirable condition is not always obtained.

We feel that any preliminary tests which may allow an estimate of the extent of these variations is most desirable. This applies particularly to material for long-time tension tests in which creep rates are determined as a function of applied stress. The time required for a statistical investigation of this kind usually precludes the possibility of making several tests under each set of conditions. As a result, much depends upon the accuracy of the assumption that each specimen has essentially the same properties as the others with which it is compared. It appears that the collection of a large amount of uniformity data is desirable in order to develop a procedure for preliminary surveys. In some of our uniformity tests we have made as many as eight Brinell tests, 40 Rockwell tests and twelve different magnetic tests on each specimen in a lot of material. The results of tests of this kind will form the basis of a later paper.

It is our practice to make a critical study of all of these uniformity data and endeavor to choose two or more specimens which appear to be most different. Tension tests of these pieces at normal temperature indicate the probable maximum variation to be expected in subsequent tests of the remaining material in the lot. We consider that this extensive preliminary investigation is justified because the interpretation of long-time tension test results requires consideration of unavoidable variations in material and testing procedure as well as the effects of the applied stress and temperature. Much time may be lost unless these variations are reduced to a minimum or are evaluated.

In reference to the possible instability at high temperatures of materials which have been hardened by heat treatment or cold work, we have made tests on cold-worked nickel which indicate that the same softening that is obtained in 5 minutes at 860° C. may be produced by heating for 4500 hours at 500° C.

Similar work on copper has been reported by Pilling and Halliwell.[2] They mention a case in which they estimate that complete softening would require 12 minutes at 300° C., 10.4 days at 200° C. and about 260 years at 100° C. **Mr. McVetty.**

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[1] Mechanical Engineer, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] N. B. Pilling and G. P. Halliwell, "Softening of Hard-Rolled Electrolytic Copper," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 97 (1925).

The data on the softening of quenched steel are not conclusive. For a particular case, Bailey[1] estimates that the softening produced in 25 minutes at 550° C. would require 5000 years at 50° C. In the case of heat-treated steels used at temperatures well below that of tempering, there are indications of gradual changes at low temperatures.

Much work has been done in studying the permanence of gages at normal temperature. Scott[2] agrees that "the decomposition of martensite will go to completion, as represented by tempering for one hour at a temperature between 400 and 500° C. (750 and 930° F.), on aging at ordinary temperatures." Although in these examples the rate of reaction is extremely slow, it is reasonable to suppose that it will be greatly accelerated in the use of hardened materials at elevated temperatures.

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[1] R. W. Bailey, "The Physical Properties of Metals and Design of Plant for High-Temperature Service," Northwestern Branch, Institution of Mechanical Engineers, England (1927).

[2] Howard Scott, "Dimensional Changes Accompanying the Phenomena of Tempering and Aging Tool Steels," *Transactions*, Am. Soc. Steel Treating, Vol. IX, p. 299 (1926).

"LONG-TIME" OR "FLOW" TESTS OF CARBON STEELS AT VARIOUS TEMPERATURES WITH PARTICULAR REFERENCE TO STRESSES BELOW THE PROPORTIONAL LIMIT

By J. J. Kanter[1] and L. W. Spring[2]

Synopsis

Since the announcement that steel will flow slowly at considerably lower loads than anticipated when at temperatures well within the present working ranges of oil refineries and steam stations, there has been much concern among engineers concerning the safety of their equipment. This naturally has resulted in extended, careful surveys of materials and of testing methods, and much thought and discussion have been given to the meaning of this phenomenon.

Since the making of "long-time" high-temperature or "flow" tests is quite time-consuming, some method of determining, or factor for estimating, rate of flow at different stresses has been sought by investigators but, seemingly, no satisfactory short-time method of estimation has yet been found.

In this paper are described twelve units of the flow testing machine with temperature controls and measuring apparatus in a laboratory making such tests. Flow curves for various carbon steels are given and discussed, as is also the effect of grain size.

Steels which have undergone small amounts of flow do not seem to have deteriorated, tensile properties including elongation and reduction of area remaining approximately what they were before the flow tests. Since little or no deterioration occurs or should be expected (unless through oxidation), and since with ductile metals at temperatures above 600° F., 10 per cent or more of flow or elongation are necessary before the material starts out on its final stretching preceding rupture, it is argued that any slight flow or deformation brought about by stresses below the proportional limit should be considered as distortion only and not seriously objectionable unless such distortion interferes with service. Usually the factors of safety employed are sufficient, but if stresses are reached which produce flow, there is little probability of rupture with danger to life or property until at least 10 or 15 per cent of total flow or distortion has taken place.

Since announcement that steel will "creep" or "flow" slowly at considerably lower loads than had been anticipated, at temperatures well within the present working ranges of oil refineries and some steam stations, there has been much concern among designing and operating engineers concerning the safety of their equipment. This naturally has resulted in extended, careful surveys of materials and of testing methods, with much thought and discussion given to the meaning of this phenomenon.

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[1] Testing Engineer, Crane Co., Chicago, Ill.

[2] Chief Chemist and Metallurgist, Crane Co., Chicago, Ill.

Many have contended that no matter what is at the bottom of it or what we call the phenomenon, the fact remains that experience with rather heavy loads and high temperatures over a period of years gives proof that the steel and other materials used are not unsafe. However, the great concern which the majority have felt is due to the fact that the formerly rather fixed pressures and temperatures have been quite rapidly raised, due, of course, to the demands which industry lately has made. Such increase is illustrated by the rather sudden advance from 250-lb. pressure with 550° F. (290° C.) temperature, to 400-lb., 600-lb., and in some cases as high as 1350-lb. pressure with total temperature of 750° F. (400° C.) in the steam industries, and to 600 or 800-lb. pressure with 800 or 900° F. (425 or 480° C.) or more in the oil-refining industries. Naturally, such increases, with larger ones demanded and withheld simply because of lack of proof that materials used in construction will be satisfactory for the service, have given considerable concern both to the fabricators and users of such equipment.

It has been somewhat disadvantageous that making tests for long-time flow required a new type of machine, and, particularly, a considerable time element for accomplishment after the testing equipment had been devised and provided. Even to-day there apparently is no long-time, high-temperature testing machine on the market specially built for the work, all those in use in the few widely-separated laboratories which are making such tests having been designed by the investigators themselves and built especially for their investigations. All of this has slowed up progress so it is not surprising that, considering the two or three years since such strict attention first was given to this matter of flow, there still is much uncertainty regarding it. Undoubtedly, it will still be a number of years before metallurgists and designing engineers can have a correct, general evaluation of the materials available for construction purposes.

Naturally, since the making of long-time high-temperature tests is so time-consuming, from three to six weeks being necessary to determine flow characteristics at a given load and temperature with fair accuracy (and even then no absolute proof that over periods of years these characteristics will remain the same), it would be highly desirable to find some quick method of determining or approximating such information. French[1] has shown quite satisfactorily that for temperatures up to 560° F. (295° C.) the load required to give long life without appreciable deformation approximates the proportional limit of the "short-time" high-temperature test, determined by very accurate extensometers. In other words, the designer could be sure that up to such a temperature, stresses less than the short-time proportional limits would be safe. There would be a very great advantage if this or some other determination that could be made within a reasonable time, approximated at the higher temperatures also the load required to bring about flow, or would indicate such flow load through some factor. However, to date there has been no agreement among those making long-time tests that at higher temperatures the proportional limit in the short-time tests is anything like a reasonably accurate measure of the flow load, or in steel bears any definite or constant relation to it. So this desideratum has not yet been found. For example, cast steel at 750° F. (400° C.) shows quite a perceptible creep at stresses approximately only one-third of the corresponding yield point and considerably below the corresponding proportional limit. (See Table I.)

TABLE I.—TESTS OF ANNEALED CAST CARBON STEEL AT 750° F. (400° C.).

(See Figs. 9 and 10)

Carbon, per cent	Tensile Strength, lb. per sq in.	Yield Point, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Rates of Flow in 500-hour Tests, in. per inch per hour		Estimated Time for 1 Per Cent Flow, years
				at 30 000 lb. per sq. in.	at 10 000 lb. per sq. in.	at 10 000 lb. per sq. in.
0.23	63 000	29 000	18 000	0.0000168	0.00000014	8.2
0.33	69 000	27 500	17 000	0.0000156	0.00000022	5.2
0.40	71 000	28 000	14 000	0.0000212	0.00000028	4.1

One not actually concerned with the making of long-time high-temperature flow determinations will hardly realize the refinements, equipment, methods, and the care necessary for accurate and authoritative results, nor the slowness with which such results are obtained. Even when results are forthcoming, definite proof of relationships, constancy, and laws governing flow of materials tested are still other steps more difficult to discover and prove. Conservatism, therefore, in the publication of results is desirable, since, until recently, comparatively little has been known as to the actual meaning of flow from either the physical testing standpoint or that of practical application.

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[1] H. J. French, "Methods of Test in Relation to Flow in Steels at Various Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 7 (1926).

CURRENT SUGGESTIONS FOR QUICK APPROXIMATION OF FLOW DATA

In the literature of flow testing so far published, very few data appear of tests made at stresses at which the deformations are small. In some of the work that has been published, the aim was to show that a "long-life" relationship exists at the yield and proportional limit stresses. Much has been proposed in the way of correlating "long-time" strength with proportional limit or other "short-time" tensile constants. The results of attempts toward this end have been rather inconclusive, setting up somewhat arbitrary criterions of the stresses at which it may be possible for flow to take place, or else criterions of flow which are argued to be low enough for general purposes. For instance, Pomp and Dahmen[1] propose and give results for a hasty determination of the flow-time curve during 2 or 3 days. This curve is repeated at two different stresses, from which they compute "rates of flow." These data then are used as the basis of a straight-line plot from which is interpolated or extrapolated the stress giving a rate of flow of 0.001 per cent per hour. This stress they define as the "long-time strength."

Just what interpretation, in terms of service, can be placed upon such a figure as this is difficult to say. It is shown by the flow-time curves of Figs. 9 to 15 that flow is continuous over considerable periods of time at a rate of 0.00001 per cent per hour, approximately one-hundredth of that determining the above-mentioned long-time strength value or so-called standard. Furthermore, the stresses producing this flow of 0.00001 per cent per hour are much lower than those which even very accurately determined elastic or proportional limit values have revealed.

With this in view, we are not so much concerned with the flow-stress relationships for the present. To the engineer the important and immediate consideration appears to be the behavior of the various materials under loads which represent feasible working conditions for whatever temperatures he has under consideration. What this procedure amounts to is a "proofing" to learn what actually happens at maximum design loads or fiber stresses, disregarding for the time being "safety factors" in the old sense, as is discussed later.

APPARATUS AND METHODS

Some of the apparatus and methods used in carrying out our high-temperature investigations, which have been going on now for more than two years, have been made known in two previous articles.[2]

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[1] *Mitteilungen* aus dem Kaiser-Wilhelm-Institut für Eisenforschung, Vol. IX, Part 3 (1927).

[2] *Power*, September 1, 1925, and February 9, 1927.

Since then, more such testing machines have been added and changes and modifications have been made in the procedures used, of which further description is given here.

These special machines were designed and built in the Crane Co. shops. An idea of the type of the machine may be had from Figs. 1 and 2. At the present time twelve of these units are in operation. The knife-edge levers of these machines are such as are used in calibrating apparatus for checking tension testing machines. The manner in which these levers are being used affords a nine-fold multiplication of the dead weight applied. The fulcrum supports and the stationary cross-head seating the self-centering socket holders into which threaded bar ends are screwed, are rigid iron castings.

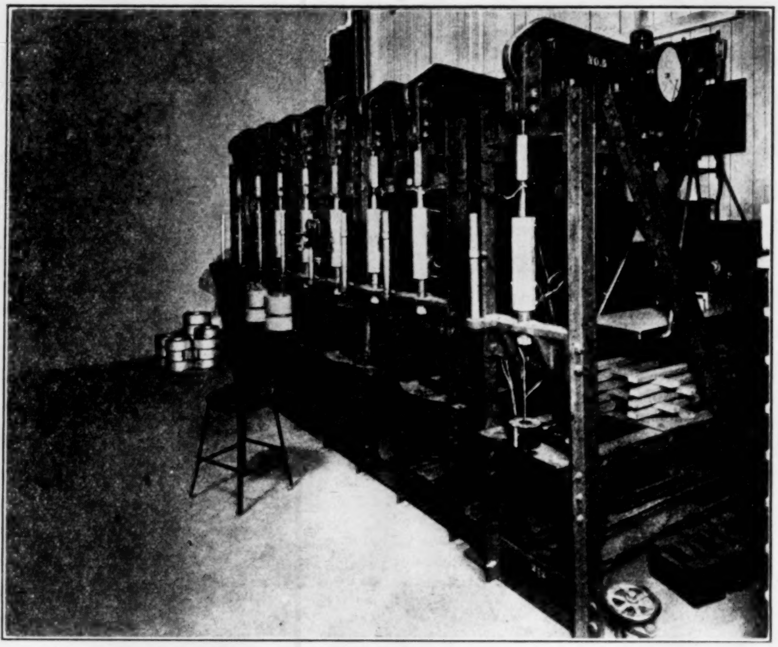


FIG. 1.—Eight of Twelve Machines in Service for the Measurement of Flow in Metals at Various Temperatures.

Loading Device.—The first five testing units constructed were built as separate machines and stand on springs placed at the four corners to absorb vibrations and shocks from the floor. The loading in the five is done with standard weights placed on bale boards hung from the levers. The last seven units were built as a multiple machine, that is, all seven units in one frame. The loading for each of these units is done with a stack of counterweights usually hung on a rod attached to the end of the beam at the rear; but for very low loads the weights are hung in front, directly from the lower end of the test specimen.

The specimens used in the high-temperature tests discussed here were turned to the standard diameter of 0.505 in. The loads required for bars of this diameter are quite readily applied since these machines are of substantial construction. By using test specimens $\frac{1}{4}$ in. in diameter it is possible to test at loads up to 250,000 lb. per sq. in. However, the diameter of 0.505 in. is used whenever possible since such bars are more representative and oxidation does not introduce as much error as with bars of smaller diameter.

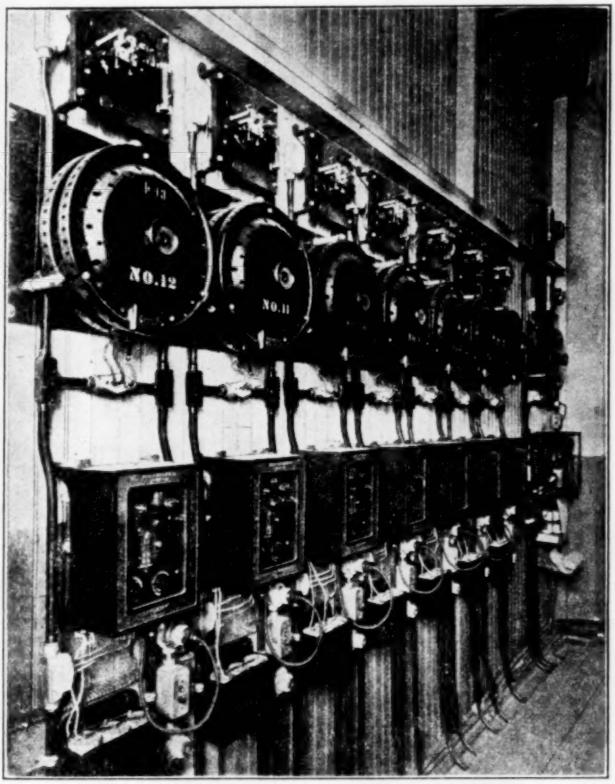


FIG. 2.—Individual Controllers, Rheostats, and Relays Used in Maintaining Constant Testing Temperatures.

On account of the simple knife-edge lever employed, friction losses in the application of the load are practically zero. In counter-balancing the levers with weights hung in the place of the bar holders to determine the exact correction necessary for its weight, it was found that the machines are sensitive to less than 0.10 lb. applied to the loading end of the lever.

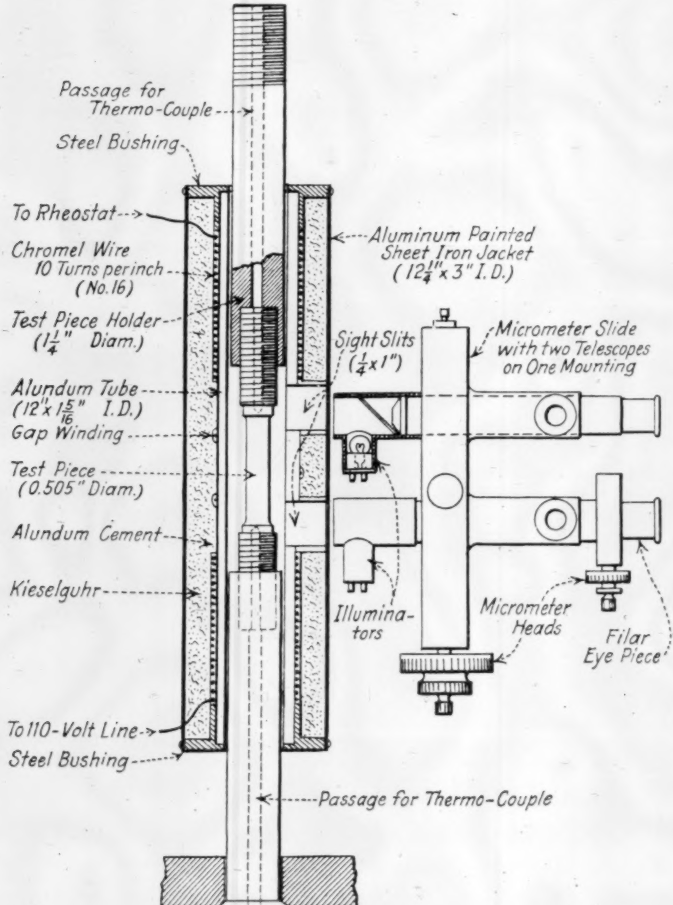

Passage for Thermo-Couple
Steel Bushing
To Rheostat
Chromel Wire 10 Turns per inch (No. 16)
Test Piece Holder (1¼" Diam.)
Alundum Tube (12" x 1 5/16" I.D.)
Gap Winding
Test Piece (0.505" Diam.)
Alundum Cement
Kieselguhr
To 110-Volt Line
Steel Bushing
Passage for Thermo-Couple
Aluminum Painted Sheet Iron Jacket (12¼" x 3" I.D.)
Sight Slits (¼ x 1")
Micrometer Slide with two Telescopes on One Mounting
Illuminators
Micrometer Heads
Filar Eye Piece

FIG. 3.—Construction and Arrangement of Furnace, Specimen and Telescopes.

Heating Control.—A gap-wound furnace, illustrated in Fig. 3, is used and has proved quite satisfactory for long continuous runs as high as 1400° F. (760° C.) without failure due to "burning out." The furnace has been carefully adjusted to give a uniform temperature over the 2-in. breaking section. The temperature is determined at all times by one or two thermocouples extending through axial holes in the holders and into the ends of the bars to within $\frac{1}{4}$ in. of the turned-down or breaking section. Careful explorations of the temperatures at various points within the test bar have shown very uniform heating and couples located in these two positions give accurate determination

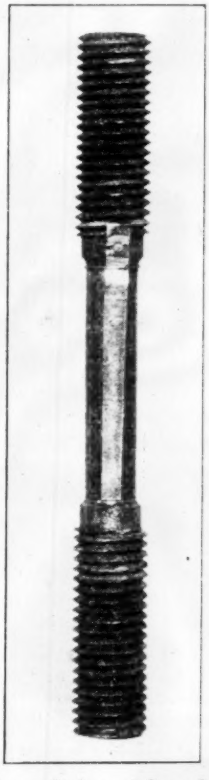
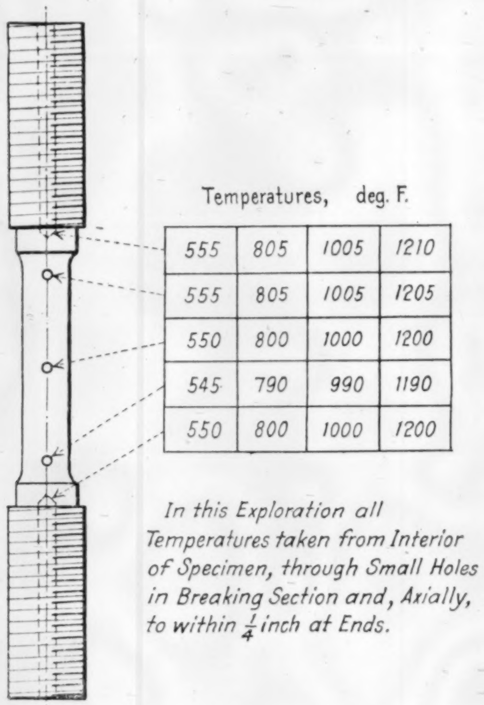


FIG. 4.—Test Bar Showing Platinum Strip and Gage Mark Wire in Place. By Means of the Filar Eyepiece, Measurement is Made of Successive Distances Between the Gage Mark Wire and the End of the Platinum Strip.

of the temperature of the bar under test (see Fig. 5). A continuous record of the temperature of the specimens in each machine is kept by means of a twelve-point Leeds & Northrup recorder. The controller couple for each machine is placed directly in contact with the winding of the furnace surrounding the specimen. Temperature control is attained by using a separate Leeds & Northrup controller for each unit, twelve controllers in all. Each controller acts through a relay which switches the current from "high" to "low." The current is adjusted through a double-plate rheostat, one plate of which is in a continuously closed circuit carrying the larger part of the current, while the other is in series with the controller, which regulates the remainder or "off and on" current necessary for close regulation of temperature. This reduces lag and results in close control, usually with not more than 5° F. (3° C.) temperature fluctuation in the test specimen.

Measurement of Flow.—In addition to accurate dead loading and close temperature control, a most important requisite is a very sensitive and positive method of observing elongation. This element of the test is met by using an optical arrangement shown in Figs. 3 and 4. This instrument consists of two telescopes mounted on a micrometer slide. The lower telescope is fitted with a filar eyepiece. Both telescopes are provided with illuminating lamps which throw light through holes in the furnace side upon the platinum gage marks. These are small platinum wires, No. 36 B. & S. gage, anchored 2 in. apart around the breaking section of the test specimen. Each testing



Temperatures, deg. F.

555	805	1005	1210
555	805	1005	1205
550	800	1000	1200
545	790	990	1190
550	800	1000	1200

In this Exploration all Temperatures taken from Interior of Specimen, through Small Holes in Breaking Section and, Axially, to within $\frac{1}{4}$ inch at Ends.

FIG. 5.—Exploration of Temperature Variation in Test Specimen Showing Temperatures at Different Points in the Test Specimen Taken Simultaneously.

machine is provided with a sturdy steel post mounted on the cross head casting allowing the telescopes to be readily attached. When a rapid rate of flow is being followed, the two telescopes are used and the gage marks are kept in view with only a small movement of the lead screws.

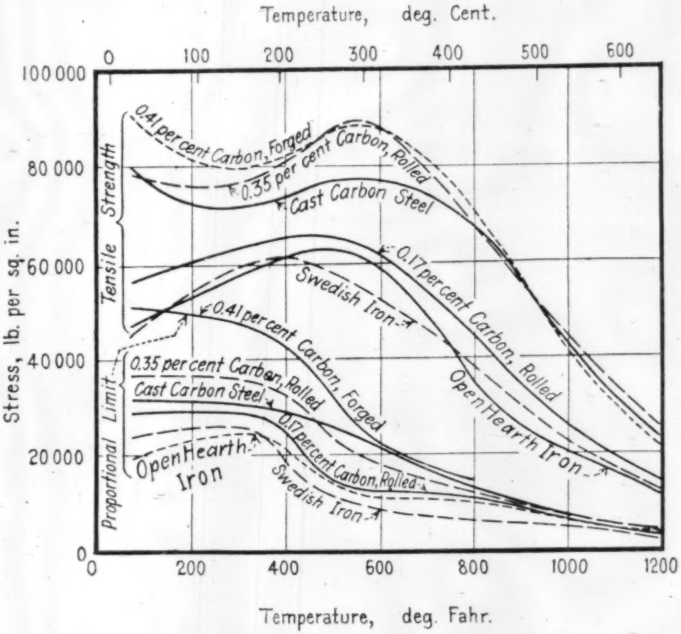


FIG. 6.—Comparison of Short-Time Tensile Strengths and Proportional Limits of the Metals Tested.

For observing the usual very minute elongations, a much more delicate modification of the optical method has been applied to these tests, permitting full advantage to be taken of the objective's resolving power. Readings reproducible to 0.00005 in. in the 2-in. gage length

TABLE II.—ANALYSES OF MATERIALS USED.

Description	Silicon, per cent	Manganese, per cent	Sulfur, per cent	Phosphorus, per cent	Carbon, per cent
Cast Steel, annealed	0.30	0.70	0.029	0.031	0.23
Cast Steel, annealed	0.34	0.77	0.029	0.032	0.33
Cast Steel, annealed	0.32	0.70	0.015	0.022	0.40
$\frac{3}{4}$-in. Open-Hearth Iron Rod	0.03	0.02	0.028	0.008	0.01
$\frac{3}{4}$-in. Swedish Iron Rod	0.02	0.02	0.004	0.053	0.03
$1\frac{1}{8}$-in. Low-Carbon Steel Rod	0.02	0.47	0.037	0.004	0.16
$\frac{3}{4}$-in. Medium-Carbon Steel Rod	0.06	0.64	0.024	0.012	0.35
Medium-Carbon Steel, forged	....	0.53	0.048	0.021	0.41

are readily obtained. For this purpose the lower telescope only is used; it is equipped with filar eyepiece, which measures the distance between the fine platinum wire fastened against the lower fillet of the test specimen and the end of a platinum strip riveted to the bar above the upper fillet, and extending low enough almost to meet the wire (see Fig. 4). Since by this scheme both ends of the specimen are brought together into the field of vision of a single eyepiece, the sensitivity attained is almost the resolving power of the objective used.

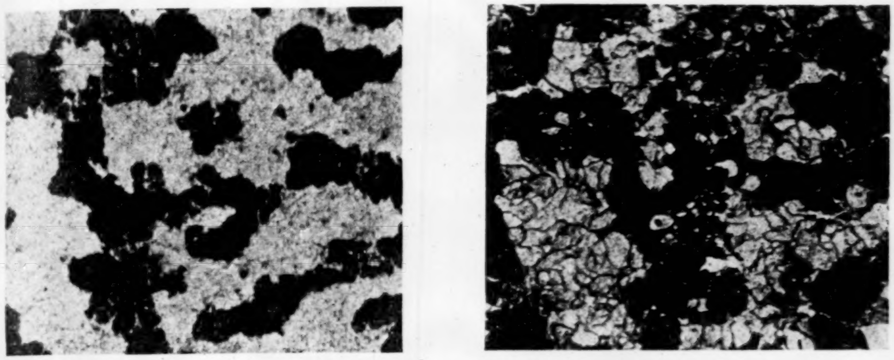
TABLE III.—SUMMARY OF COMPARATIVE DATA DERIVED FROM VARIOUS FLOW-TIME CURVES.

Material	Flow Test Results				Short-Time High-Temperature Results			References to figures illustrating the long-time data given
	Testing Load, lb. per sq. in.	Total Flow in 500 Hours, in. per inch	Secondary Rate of Flow, in. per inch per hour	Estimated[a] Time for 1 Per cent of Total Flow	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	
TESTED AT 800° F. (425° C.)								
Open-Hearth Iron, normalized	10 000	0.01320	0.0000252	15 days	10 000	15 100	35 100	Fig. 13
Open-Hearth Iron, water quenched	10 000	0.01040	0.0000215	20 days	......	......	......	Fig. 13
Swedish Iron, normalized	10 000	0.02150	0.0000363	8 days	6 000	14 000	38 900	Fig. 13
0.17-per-cent Carbon Steel, normalized	10 000	0.00056	0.00000026	4.2 yrs.	11 500	20 500	44 600	Figs. 13, 14
0.35-per-cent Carbon Steel, normalized	10 000	0.00065	0.00000017	6.4 yrs.	12 500	31 000	67 100	Fig. 15
0.35-per-cent Carbon Steel, annealed	10 000	0.00034	0.00000016	7.0 yrs.	14 000	24 400	59 800	Fig. 15
0.35-per-cent Carbon Steel, quenched	10 000	0.00065	0.00000021	5.1 yrs.	29 000	52 000	70 300	Fig. 15
0.41-per-cent Carbon Steel, as forged	10 000	0.00054	0.00000014	7.7 yrs.	14 000	30 200	70 200	Fig. 14
0.40-per-cent Carbon Cast Steel, annealed	10 000	0.00074	0.00000022	4.8 yrs.	12 000	27 000	67 000	Fig. 14
0.33-per-cent Carbon Cast Steel, annealed	10 000	0.00022	0.00000015	7.6 yrs.	16 000	26 000	65 400	Fig. 14
TESTED AT 1000° F. (540° C.)								
Open-Hearth Iron, normalized	3000	0.00096	0.00000053	2.0 yrs.	7 000	14 000	20 200	Fig. 16
0.17-per-cent Carbon Steel, normalized	3000	0.00085	0.0000005	2.0 yrs.	6 300	14 500	26 110	Fig. 16
0.35-per-cent Carbon Steel, normalized	3000	0.00070	0.00000024	4.5 yrs.	7 000	23 200	45 750	Fig. 16
0.35-per-cent Carbon Steel, quenched	3000	0.00136	0.00000085	1.2 yrs.	11 000	28 900	43 400	Fig. 16
0.41-per-cent Carbon Steel, as forged	3000	0.00195	0.00000297	0.4 yrs.	......	26 500	39 850	Fig. 16
0.33-per-cent Carbon Cast Steel, annealed	3000	0.00053	0.00000021	5.5 yrs.	......	20 000	45 000	Fig. 12
0.40-per-cent Carbon Cast Steel, annealed	3000	0.00079	0.00000095	1.2 yrs.	7 500	20 000	45 000	Fig. 12
TESTED AT 1200° F. (650° C.)								
Open-Hearth Iron, normalized	500	0.00117	0.0000016	0.7 yrs.	3500	5 700	11 100	Fig. 17
0.35-per-cent Carbon Steel, normalized	500	0.00210	0.0000040	0.3 yrs.	4000	13 500	26 200	Fig. 17

[a] Estimated on basis of secondary rate of flow.

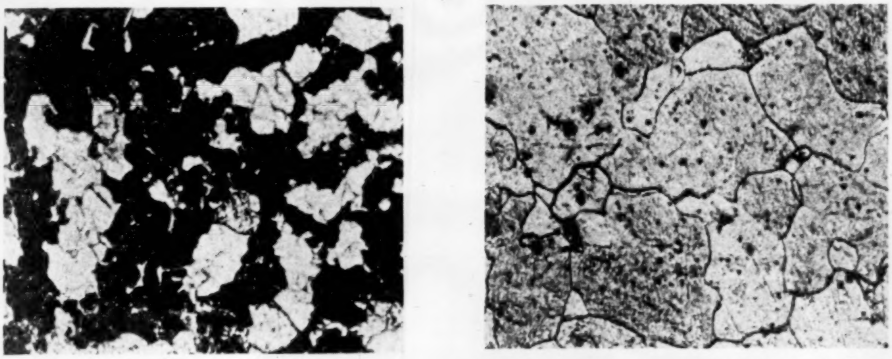
MATERIALS TESTED

The data presented in this paper are confined entirely to low-carbon steels and irons, although much work has been done on alloy steels and other materials. In Table II are given the analyses of the materials used in the tests referred to in the figures, tables and text. The complete short-time tensile strength and proportional limit curves



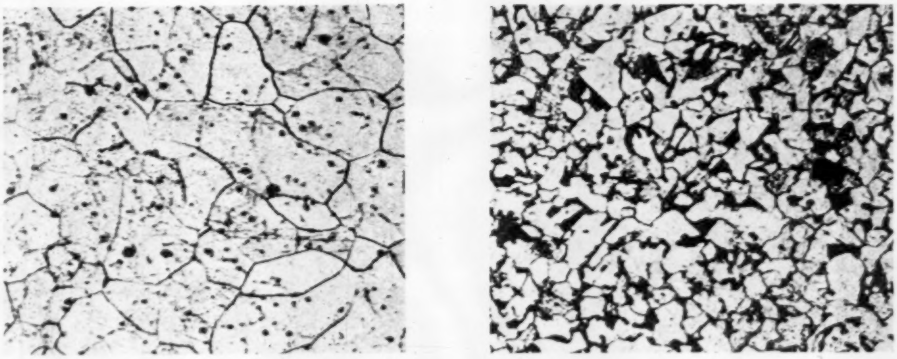
(*a*) 0.23-per-cent carbon cast steel, annealed.

(*b*) 0.33-per-cent carbon cast steel, annealed.



(*c*) 0.40-per-cent carbon open-hearth cast steel, annealed.

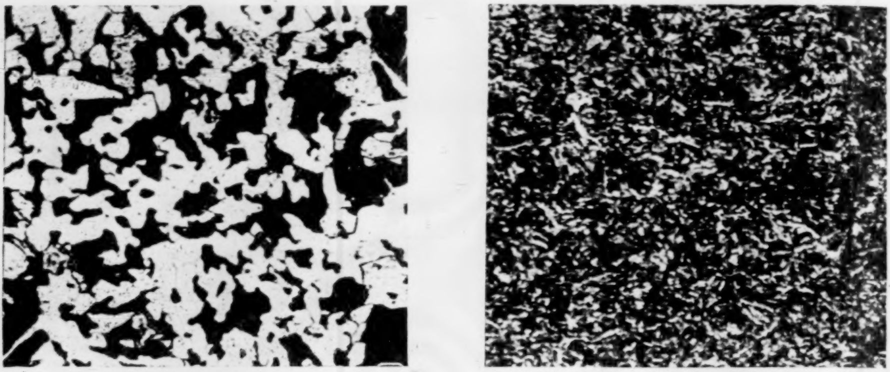
(*d*) Open-hearth iron, normalized from 1800° F. (980° C.).



(*e*) Swedish iron, normalized from 1800° F. (980° C.).

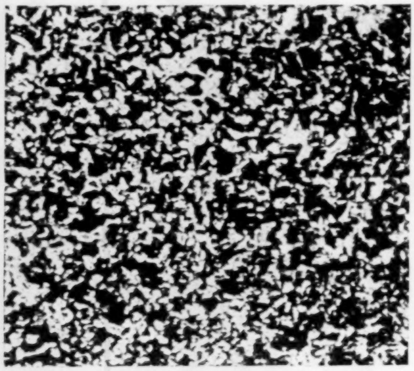
(*f*) 0.17-per-cent carbon rolled steel, normalized from 1650° F. (900° C.).

FIG. 7.—Microstructures of the Metals as Tested. Etched in 5-per-cent Nitric Acid (× 100).



(*g*) 0.35-per-cent carbon rolled steel, annealed at 1550° F. (845° C.).

(*h*) 0.35-per-cent carbon rolled steel, quenched from 1550° F. (845° C.), drawn at 1000° F. (540° C.).



(*i*) 0.41-per-cent carbon steel, forged to 1 by 1 in. from 4 by 6 in.

FIG. 7. (*Continued*).—Microstructures of the Metals as Tested. Etched in 5-per-cent Nitric Acid (× 100).

from 70 to 1200° F. (20 to 650° C.) for each of the materials covered are shown in Fig. 6. Photomicrographs of some of the structures are given in Fig. 7, while in Table III will be found a summary of comparative data derived from the various flow-time curves of tests made at 800, 1000, and 1200° F. (425, 540, 650° C.).

FLOW-STRESS-TEMPERATURE RELATIONSHIPS

In Figs. 10, 13 and 14 are given certain flow-time curves for steel at 750 and 800° F. (400 and 425° C.), using a stress of 10,000 lb. per sq. in. This stress was chosen from consideration of the requirements

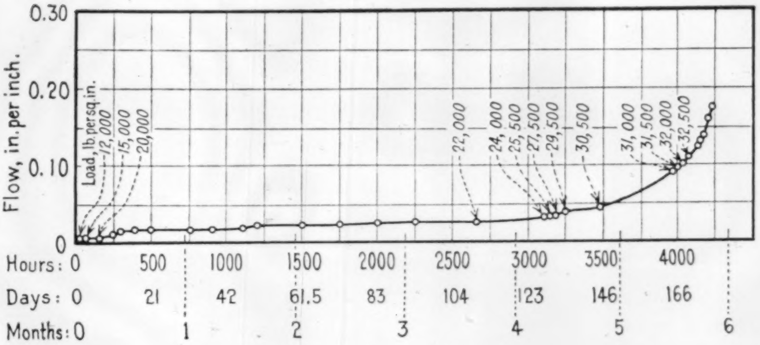


FIG. 8.—Flow-Time Curve of 0.23-per-cent Carbon Cast Steel, Tested at 800° F. (425° C.). In this Six-Months' Continuous Test (4226 Hours), Test Specimen Was Loaded at 20,000 lb. per sq. in. for 2437 Hours Following Which Several Increases in Load Hastened Final Stretch and Rupture.

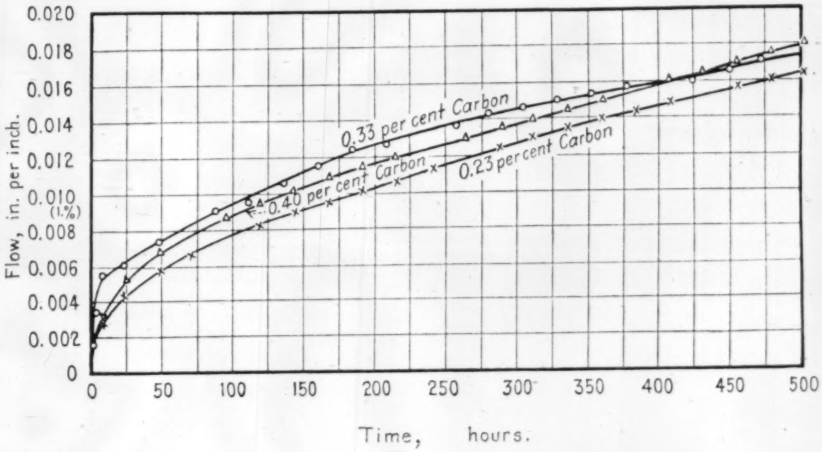


FIG. 9.—Flow-Time Curves for Annealed, Cast Carbon Steels at 750° F. (400° C.) Under Loads of 30,000 lb. per sq. in.

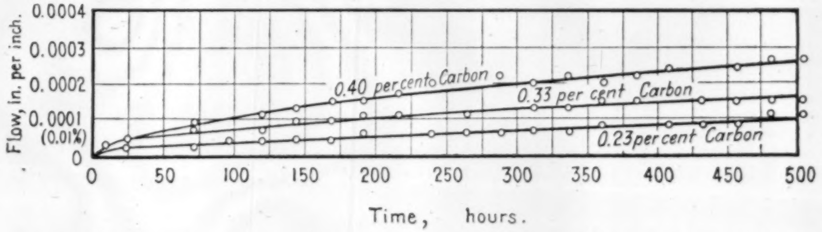


FIG. 10.—Flow-Time Curves for Annealed, Cast Carbon Steels at 750° F. (400° C.) Under Loads of 10,000 lb. per sq. in.

which some types of design make upon this material. For purpose of comparison at these temperatures it may be considered representative. For similar reasons, 3000 lb. per sq. in. was chosen for the comparison at 1000° F. (540° C.) (see Fig. 16). The behavior of steels in flow under these conditions, therefore, determines their suitability for certain types of existing practice.

Semi-Logarithmic Flow-Time Curves:

In the case of cast steel, the flow-time relationships have been determined at enough different stresses to plot curves for stress *versus* rate of flow at constant temperature. Figure 18 is given to show the approximately straight-line relationship that exists when the time estimated to produce 1 per cent of flow is plotted against load upon semi-logarithmic coordinates. By extending the number of "log scales" to include the periods of time involved in short-time testing, the tensile strengths and yield points have been placed on the diagram. It is interesting to note that the approximately straight lines passing through the flow-stress points, when extrapolated, pass through the points marking tensile strength determined by short-time test. Just what significance, if any, should be attached to this observation we cannot say at this time. Although there is little logical basis for it, it might be inferred from these curves that the tensile strength at high temperatures might be almost indefinitely reduced by slow enough separation of the testing machine cross-head since it is well known that the rate of loading during the plastic deformation in a short-time tension test does exert a considerable influence upon the tensile strength value. It might be argued that the yield stress also should fall on the curve at the ordinate corresponding to the interval of time required to produce 1 per cent of elongation defined in a suggested "standard method" as yield point.[1] This it does not do at lower temperatures, though at the higher temperatures represented the yield stress does fall rather close to the curve.

Stress-Temperature Curves:

From such flow-stress, constant-temperature curves, the curves shown in Fig. 19 have been constructed in comparison with "short-time" tensile strength curves for the same material, carbon cast steel. Each of these curves, for any temperature, gives the stress estimated to produce in the metal 1 per cent of flow in the designated period of time. To distinguish these curves from other temperature-strength

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[1] Tentative Methods of Tension Testing of Metallic Materials (E 8 – 27 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1067 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 725.

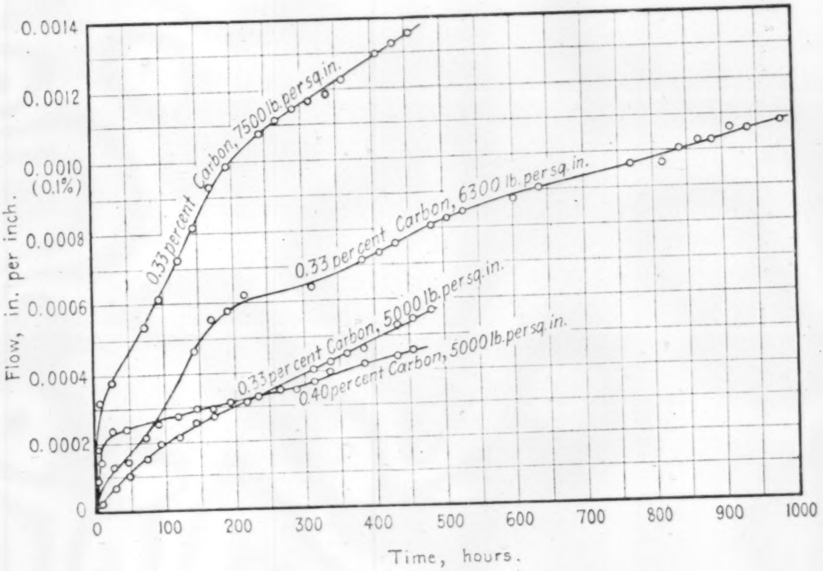

FIG. 11.—Flow-Time Curves for Annealed Cast Carbon Steels at 900° F. (480° C.) Under Different Loads.

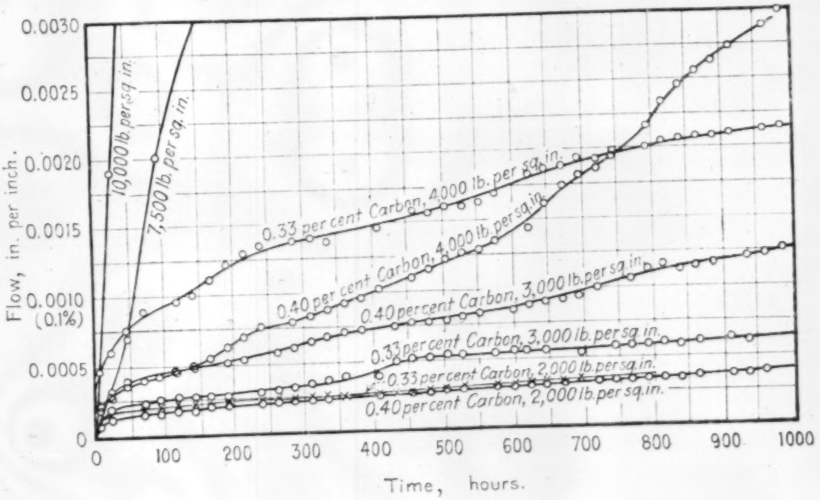

FIG. 12.—Flow-Time Curves for Annealed Cast Carbon Steels at 1000° F. (540° C.) Under Different Loads.

curves, such as tensile strength, yield point, proportional limit, "creep limit," etc., it has been suggested that they be called "iso-flow-rate" curves. It will be noted that the whole family of these curves shown, representing periods of from one hour to ten years, exhibits similar shape to the tensile strength curve and tends in the same general direction. The yield point and proportional limit curves are crossed and under cut by the iso-flow-rate curves. A series of such curves is of value to the designer, as they afford him a wider scope from which to consider the material and he can work to the flow curve which best fulfills his particular requirements.

The sense in which "rate of flow" is used in speaking of these data needs some qualifications. It is obvious, of course, that none of the tests have been carried out for a sufficient length of time to establish beyond dispute exactly what kind of a function rate of flow is. At the small loads that are considered here, there is a tendency for the curves to approach a horizontal asymptote after several hundred hours of testing, even at 1000° F. (540° C.). Whether or not there would be a complete cessation of flow in some cases if the tests were prolonged for periods of time reaching into years, is highly speculative. Therefore, the interpretation placed upon rate of flow is a conservative one. Although the secondary portion of the flow-time curve does not seem to be a linear function, it does serve at least to fix an upper limit to

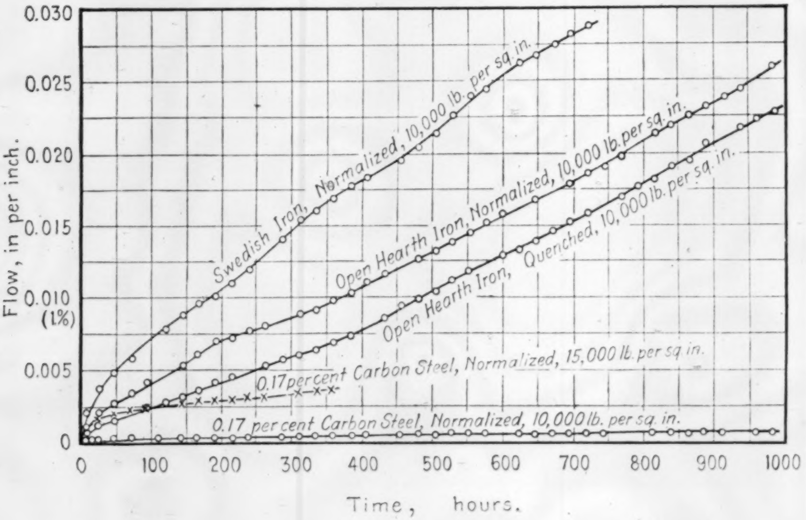

FIG. 13.—Flow-Time Curves for Two Carbonless Irons and 0.17-per-cent Carbon Steel at 800° F. (425° C.), all Loaded to 10,000 lb. per sq. in. with Additional Specimen of 0.17-per-cent Carbon Steel Loaded to 15,000 lb. per sq. in.

what the subsequent rate of flow may be. For this reason estimates of deformation based upon several hundred hours of testing after the comparatively rapid primary flow period, are conservative.

REGARDING LIMITING CREEP STRESSES

Some investigators of flow or creep in metals at high temperatures seem to have shown that comparatively large loads are sustained without appreciable flow after initial flow has taken place for several weeks. This behavior is stated to have been found even at such a high temperature as 1000° F. (540° C.). However, when the load is increased beyond a certain maximum, there is no cessation of flow as before, but, instead, flow is continuous until fracture. It is to be noted that near the lower limit of the stress range where flow may seem to have ceased, the total flow is equivalent to 2 or 3 per cent elongation, this in a period of not more than from three to six weeks. Obviously, loads and temperatures producing even this much deformation in such short periods of time are not satisfactory as bases for design values in structures which are expected to show no distortion over a period of many years. Even though ultimate failure from rupture might not occur for an indefinitely long time, a few per cent of flow may render equipment or a structure useless.

This raises the question, how are we to know the permissible stresses producing only fractions of a per cent of flow in a year? Certainly not from "creep limits" at which enough flow can take place in several weeks to warp a high-temperature plant out of usefulness. The element of uncertainty in using data such as these is fully as great as in using the proportional limit or yield point, and they could be employed only by using arbitrary safety factors. This, it seems, would be no great advance in the engineering of plants or structures for high-temperature service.

Inasmuch as flow can be demonstrated to take place at stresses far below the so-called flow limit at 1000° F. (540° C.), for the purposes of precise engineering it should be advisable to concentrate attention upon the behavior of metals in flow at those stresses at which the initial flow does not transcend practical proportions before a stable stage is reached, but that give a negligible amount of initial flow. This we have aimed to do.

Demonstrating the above, a number of flow-time curves for various temperatures are given to show the comparison between steels of various carbon contents and heat treatments. Figures 13, 14 and 16 show that open-hearth iron and Swedish iron flow much more rapidly at 800° F. (425° C.) and 10,000 lb. per sq. in. than do steels of 0.17, 0.35, and 0.40 per cent carbon, while at 1000° F. (540° C.) and 3000 lb. per sq. in. the steels show no particular advantage over the irons. At 1200° F. (650° C.), with 500 lb. per sq. in., the open-hearth iron (Fig. 17) seems to resist flow somewhat better than the 0.35-per-cent carbon steel.[1]

INFLUENCE OF CARBON AND GRAIN SIZE ON FLOW

With reference to the effect of carbon content on the flow of steel, we must bear in mind the relationship between flow and tensile strength. The data on short-time tensile strength of the steels tested in flow are given in Fig. 6. This shows greater tensile strength and yield point for increased carbon content, even at 1000° F. (540° C.). This is contrary to what might be inferred from the behavior in flow under small stresses. These facts would make it appear that in so far as carbon is responsible for increased strength (as in the 800° F. (425° C.) tests) it increases resistance to flow. At the lower temperatures, where small grain size is a prime factor governing the strength of steel, carbon increases strength since refinement accompanies its presence. At low temperatures, deformation in a steel with small

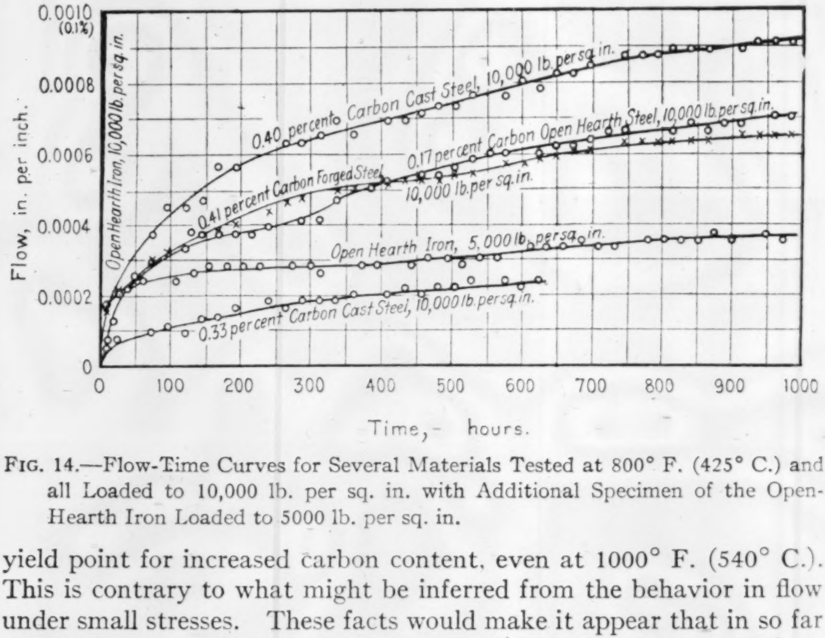

FIG. 14.—Flow-Time Curves for Several Materials Tested at 800° F. (425° C.) and all Loaded to 10,000 lb. per sq. in. with Additional Specimen of the Open-Hearth Iron Loaded to 5000 lb. per sq. in.

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[1] At the higher temperatures, the higher-carbon materials have greater tensile and elastic limit strengths than the lower-carbon materials (see short-time curves Fig. 6), thus providing greater factors of safety and security than those which the lower-carbon materials would provide. On the other hand, at temperatures above 1000° F. (540° C.) the carbonless and lower-carbon materials do seem to resist slightly better the slow flow conditions than do the higher-carbon materials. However, as shown on page 101, flow and tensile properties (including elongations) have separate engineering valuations, and short-time stiffness or strength are valuable, as always.

grain is accompanied by great strain-hardening, and continuous flow sufficient to rupture is observed only at loads approaching the tensile strength. At higher temperatures (above 800° F. (425° C.)) it is different; the grain size is still small, it is true, but the grains themselves are no longer as obdurate and brittle as at the strain-hardening temperatures; instead they are more plastic and capable of changing

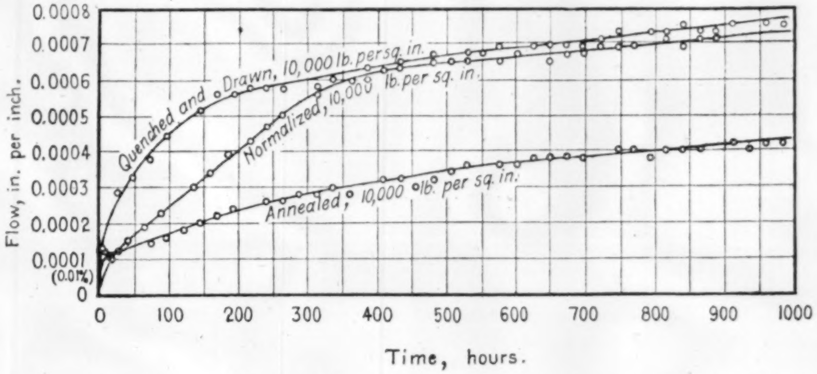

FIG. 15.—Flow-Time Curves for 0.35-per-cent Rolled Carbon Steel, Showing Effect of Various Heat-Treatments, Under Loads of 10,000 lb. per sq. in. at 800° F. (425° C.).

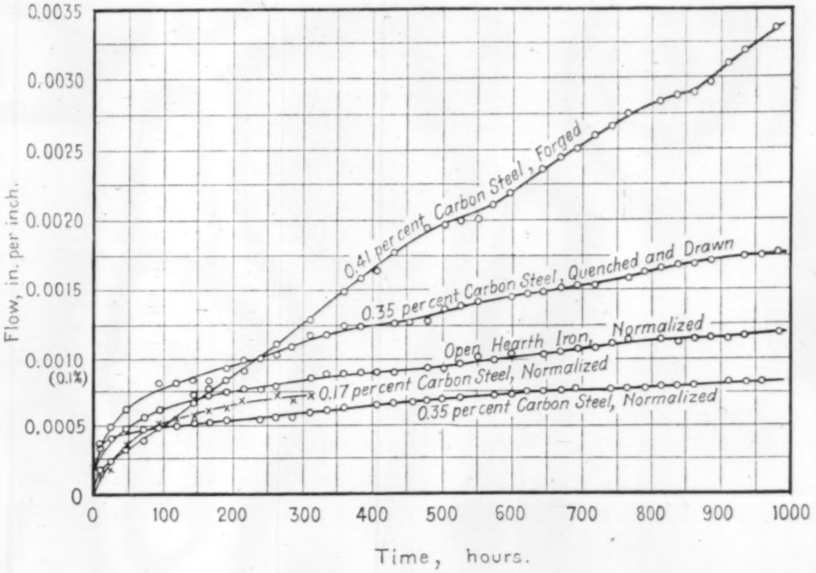

FIG. 16.—Flow-Time Curves for Several Materials Under Loads of 3000 lb. per sq. in. at 1000° F. (540 °C.).

form under stress without splitting and forming the well-known slip bands, with more grains to obstruct further slipping. In this plastic condition, strength by mere virtue of number or multiplicity of grain does not obtain. Thus the grain-size theory of hardness or strength seems to be consistent with the fact of easy continuous flow when temperature mounts.

As has been pointed out, carbon content counts for less and less as the temperature rises. Hence, so far as increasing strength and inhibiting flow is concerned, small grain size becomes of little or no advantage; in fact, the more rapid flow of the 0.35-per-cent carbon steel over the open-hearth iron at 1200° F. (650° C.) would indicate that possibly large grain is really an advantage at such temperature.

Flow Curves and the Slip-Interference Theory:

In an attempt to test the consistency of the foregoing conception of influence of grain size on flow, some samples of the open-hearth iron and 0.35-per-cent carbon steel were heat-treated to bring about different grain size so that direct comparisons of flow-time curves could be made. Unfortunately, the tests have not progressed far enough at the time of this writing to furnish very definite conclusions; however, a few of the curves are given in Figs. 13, 15, and 16. It will be seen upon comparing the photomicrographs (Fig. 7) and flow-time curves, that the smaller the grain size the more rapid the flow, particularly in the initial stage. This phenomenon is singularly interesting from the standpoint of the slip-interference theory of hardness, since it perhaps offers a means of checking up on the validity of certain ideas as to why flow differentiates into the primary and secondary stages.

Flow in a metal, under stress, might be conceived of as being due to a continuous distortion taking place within an aggregate of differently oriented crystals. The effect observed is an elongating in the direction in which the force is applied. It seems quite obvious that the way in which flow takes place is largely through slippage in the crystal lattice, this mechanism of deformation in metals being a rather well-established theory.

In order to explain or differentiate initial and secondary flow upon the slippage theory, an assumption can be made relative to the behavior of the crystal aggregate. Assuming, due to differences of position and orientation, that there exists a tendency for some crystals to slip more than others under the force applied, adjustments must occur at the grain boundaries at places where the more rapidly slipping crystals adjoin those slipping less rapidly. Adjustment then takes place rather quickly at such places along the grain boundaries where the components of the acting forces are large. The adjusting process continues until equilibrium is reached between slip in the crystals and movements along grain boundaries. Hence the primary flow is assumed to be composed of much boundary movement plus slip, while secondary flow is largely slip in the crystal alone. An immediate correlary that comes out of this hypothesis is that the grain size of a metal will influence or control the amount of initial flow, since, when large grains are being dealt with, the boundary movements assume smaller proportions. By this is meant, merely, that the grain boundary area to a given volume of metal is less when the grains are large. This concept offers a quantitative means of testing the validity of the foregoing assumptions about the mechanism of flow.

FLOW FROM A PRACTICAL STANDPOINT

However, to return to the engineering aspect, certain things are becoming evident, one of these being that the great fear and the considerable gloom which has enveloped the metallurgist and designing engineers concerned with this matter have been considerably too deep. Remark has been made in various quarters that if conditions were as bad as stated, many of our pipe lines and other parts of equipment working at high temperatures would not be standing up under the loads as they are at present. This, presumably, is true. Answer should be made that operating pressures are not as severe as the testing loads that have produced flow in the laboratory, since factors of safety have been sufficient, though undoubtedly lower than presumed; moreover, very often the temperature actually may not have been quite as high throughout the section of metal as was supposed.

The engineer for a long time considered steel as having a tensile strength of 60,000 lb. per sq. in. with yield point of 30,000 lb. per sq. in. Later, from short-time high-temperature tests, he realized that at 1000° F. (540° C.), say, the tensile strength is only approximately 30,000 lb. per sq. in. and the yield point 15,000 lb. per sq. in. Now that it is admitted that at the same 1000° F. (540° C.), steel will "flow" or "creep" under a stress of only 3000 lb. per sq. in., he naturally is troubled. Having dealt for so long with yield points as the stress at which steel test specimens in the tension testing machine elongate and with tensile strength as the stress at which the specimens break within a few minutes, he has brought this concept into the flow-stress idea and very naturally has associated the two. The result, of course, is his assumption that with steel "flowing" at 3000 lb. per sq. in.

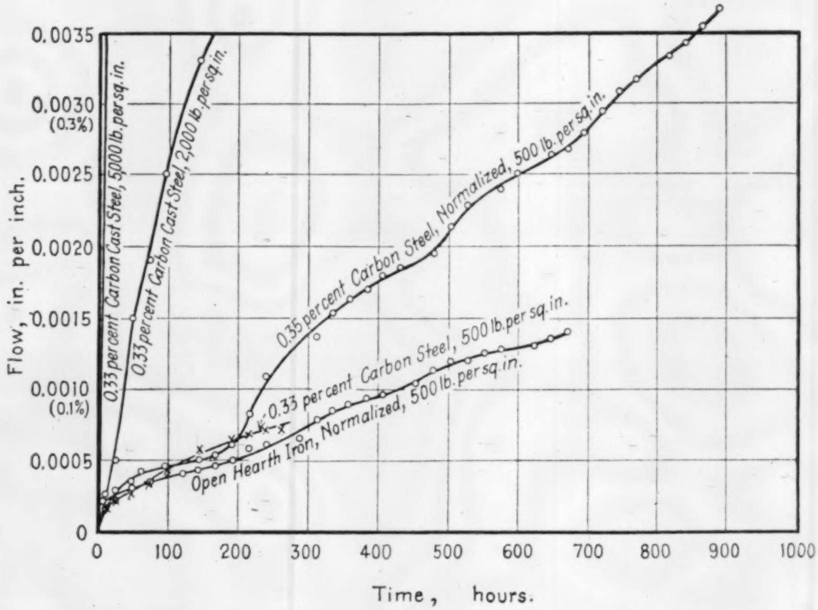

FIG. 17.—Flow-Time Curves for Several Materials Tested at 1200° F. (650° C.). Open-Hearth Iron is Shown to Flow at a Slightly Lower Rate Under 500 lb. per sq. in. than 0.35-per-cent Carbon Steel. (See footnote on page 98.)

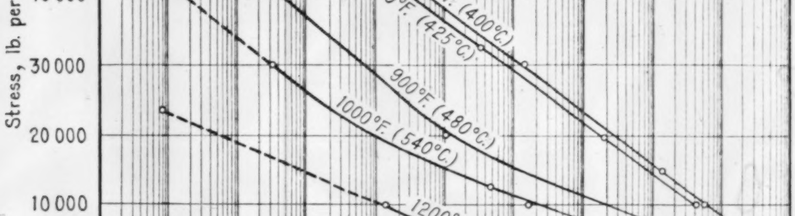

FIG. 18.—Relationship Between Load and Time to Produce 1 per cent of Flow in Annealed Cast Carbon Steel at Different Temperatures.

When short-time tensile strengths are plotted against the time ordinate for 1 per cent of flow at pulling speed (1 per cent in 3 seconds) and are connected to the load-time graphs, the curves appear continuous.

there is no factor of safety and the material is likely at any time to rupture with consequent hazards.

The facts seem to be that the stretch or elongation at stresses above the elastic limit and the very slow flow which occurs at much lower stresses over a long time, are far different. From short-time high-temperature tests (Fig. 6) it will be noted that at 1000° F. (540° C.), the tensile strength and yield point are approximately 30,000 and 15,000 lb. per sq. in., respectively. The flow-producing stress of 3000 lb. per sq. in. reported on this material as giving 0.0000005 in. per inch per hour flow (1 per cent in two years) brings about an extremely slow distortion in proportion to the load, which, so far as bringing about any early rupture of the material is concerned, is practically negligible. To bring about rupture in a short period of time, the stress would have to be increased to somewhere near the short-time yield point, where the steel under observation would start visibly to elongate; but it would require stresses near to the short-time tensile strength to bring about rupture within any comparatively short time.

There are, then, these two different and quite separate concepts—the very slight flow or creep produced by low loads and the much greater elongation which occurs at stresses around or above the yield point. It appears that this fact should increase considerably the peace of mind of every one concerned. As shown in the figures obtained through careful tests in long-time testing equipment, a certain amount of flow is produced even with rather low loads, and, in a general way, the rate of flow is proportional to the amount of load. A rate of flow of 0.000001 in. per hour per inch of length of specimen is equivalent approximately to 0.01 in. per inch per year, or 1 per cent stretch. This can be, and we believe should be, considered largely as distortion. That is, should proper factors of safety be neglected and a tank, pipe, valve, or other apparatus be subjected continuously to high enough temperature and sufficient pressure be built up to produce circumferential flow or stretch of 0.01 in. per inch per year, it would result simply in the slight expansion of the tank, pipe, or valve; and unless such increase of size interferes with operation or serviceability by making the apparatus take up too much room, throwing parts of the equipment out of line, etc., it would do little or no harm, certainly no harm so far as safety to life or plant is concerned. There is no danger of rupture for a long period of time unless for some reason flow is seriously localized through overheating at some point where it is not detected, as, for example, in a boiler or superheater-tube.

Tests made by the authors, as well as those reported by others, show that test specimens of ductile metals at temperatures of more than 600° F. (315° C.) and at stresses that produce flow, do not rupture until warning has been given through stretch or "flow" of 10 to 25 per cent. This is distinctly parallel to the test specimen in the short-time tension test which likewise elongates 10 to 25 per cent, both "necking down" before rupture. With this in mind, danger to life through failure by deformation from flow of such small magnitude that the total extension does not disturb serviceability over a period of years, may be dismissed as practically impossible, and the whole matter of useful life can be considered mainly from the standpoint of distortion per year or other suitable period, and whether such distortion is troublesome.

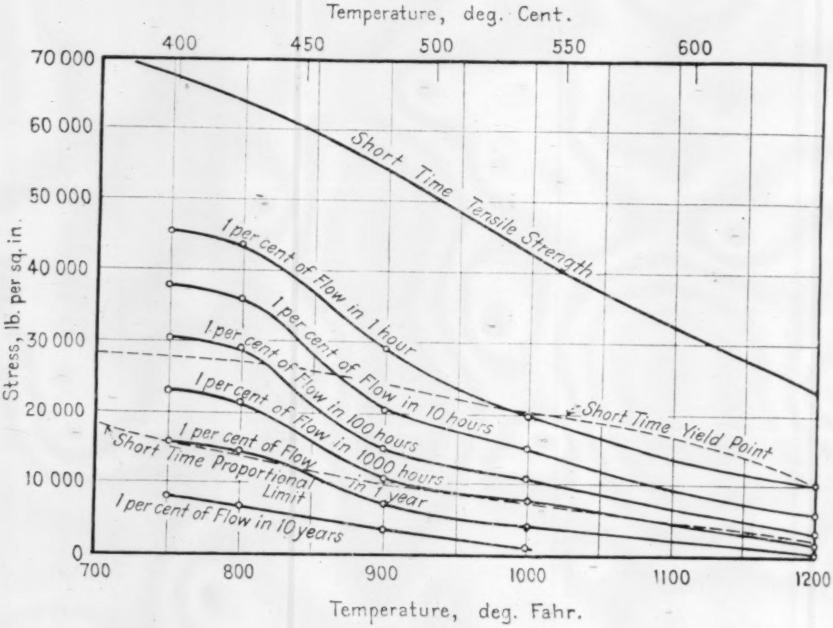

FIG. 19.—"Iso-Flow-Rate" Curves for Carbon Cast Steel with Short-Time Tensile Strength, Yield Point, and Proportional Limit Curves Drawn in for Comparison.

It is realized that the tests here given are of much shorter duration than that required of similar materials in service. Table IV indicates that little or no deterioration has occurred during the period of testing since test specimens which had been subjected to a period of temperature and flow gave practically the same tensile

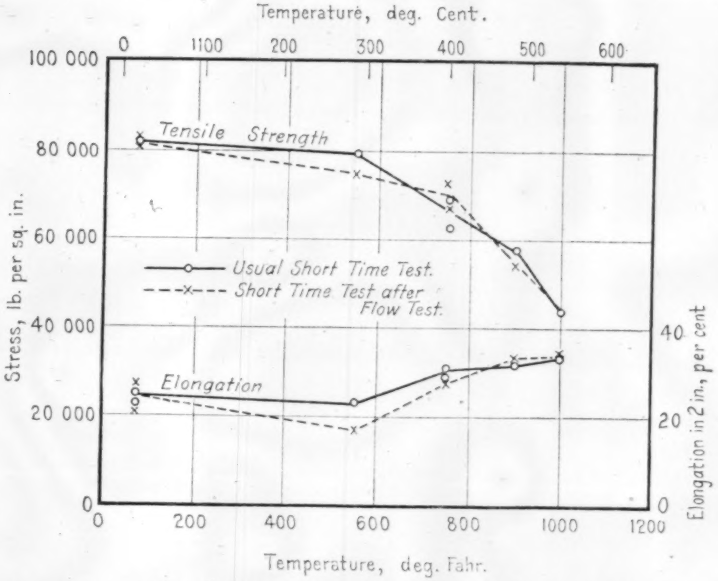

FIG. 20.—Cast Carbon Steels Do not Seem to Be Deteriorated by Conditions of Stress, Time and Temperature Producing Flow. Short-time tensile strength and elongation curves shown by dotted lines were determined at various temperatures after the specimens had been subjected to flow test and removed unbroken from the flow test machines. The solid lines represent new steels. (See Table IV for individual results.)

TABLE IV.—SHORT-TIME TESTS ON CAST STEEL SPECIMENS PREVIOUSLY TESTED IN FLOW.

(See Fig 20 for graphic representation.)

Material	Flow Test History					Subsequent Short-Time Tests						
	Temperature, Long-Time Test		Time Under Tests, hours	Time of Testing Loads, hours	Total Flow, in. per inch	Temperature, Short-Time Test		Tensile Strength, lb. per sq. in.	Yield Point, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent
	deg. Fahr.	deg. Cent.				deg. Fahr.	deg. Cent.					
0.33-per-cent Carbon Steel	750	400	1008	30 000	0.0400	70	20	82 900	53 800	51 800	21.0	34.0
	1000	540	1008	4 000	0.00216	70	20	80 450	43 000	39 000	27.5	41.5
	....	...	none	.....	none	70	20	83 050	43 500	39 500	25.0	38.5
0.40-per-cent Carbon Steel	900	480	504	4 000	0.00045	70	20	82 400	42 000	41 000	23.5	36.2
	....	...	none	.....	none	70	20	81 100	37 800	36 000	24.0	34.7
0.40-per-cent Carbon Steel	550	290	456	12 000	0.00031	550	290	74 700	29 300	21 500	23.5	27.0
	....	...	none	.....	none	550	290	80 000	30 000	19 000	17.0	25.0
0.23-per-cent Carbon Steel	750	400	1100	30 000	0.0160	750	400	67 500	39 400	29 500	27.5	42.4
	....	...	none	.....	none	750	400	63 000	29 000	18 000	29.0	50.0
0.33-per-cent Carbon Steel	750	400	1672	15 000	0.00090	750	400	72 900	28 500	18 000	28.0	42.7
	....	...	none	.....	none	750	400	69 000	27 500	17 000	31.0	53.0
0.40-per-cent Carbon Steel	900	480	504	5 000	0.00047	900	480	54 600	23 700	14 000	33.5	55.4
	....	...	none	.....	none	900	480	58 000	25 000	10 000	32.0	59.0
0.33-per-cent Carbon Steel	1000	540	1008	3 000	0.00130	1000	540	45 100	21 000	9 000	34.0	54.1
	....	...	none	.....	none	1000	540	45 000	20 000	8 000	33.5	59.0

properties as duplicate specimens without any previous testing. Deterioration should not be expected unless through oxidation, which can be fairly well prevented.

As has been pointed out by others, higher pressures have comparatively less effect than higher temperatures in so far as approach to the tensile strength or yield point of materials is concerned. It has been realized from the first that for higher pressures, heavier pipe, valves, fittings, and other pressure vessels should be used. To take care of the flow at high temperature, the means of relief should be the same. In the mechanical world, enormously heavy work and difficult jobs are done simply by the use of sufficiently powerful engines, cranes, etc. The same should answer here—heavy enough equipment. This, therefore, argues against any tendency to make thinner the walls of pipes, valves, fittings and pressure vessels. Such practice may be unsafe not only from the standpoint of giving equipment too light to withstand flow stresses, but also from the standpoint of corrosion, which in some applications of to-day may be more serious than we think. The oil industry recognizes this as a powerful factor, because of the corrosive sulfur compounds encountered in the "cracking" process. Possibly with the higher pressures and temperatures of superheated steam operation, chemical reactions in the steam boiler, not recognized to-day, may be found later to be serious. For this reason, also, sufficiently heavy design is advisable.

DISCUSSION

Mr. French.

MR. H. J. FRENCH[1] (*presented in written form*).—The results on cast steels reported by Messrs. Kanter and Spring show good agreement with those obtained on wrought steels of somewhat similar composition by the writer and some of his associates. This is gratifying since it was shown some time ago[2] that wide differences were encountered in the results of flow tests procured by different investigators just as they have been encountered in the short-time tension tests.

Figure 19 in the paper should be very helpful in the practical application of the results since a graphic representation between stress, temperature, deformation and time enables the designer to make at least an intelligent estimate of allowable working stresses for varying life-deformation requirements of service.

A similar method of summarizing the results of flow tests was used by the writer in a paper prepared about a year ago.[2] So-called "creep charts" were prepared which showed the stresses producing different amounts of deformation per thousand hours at different temperatures. Such charts for five wrought steels are reproduced in the accompanying Fig. 1.

Few investigators have been able to prepare similar summaries, as tests of relatively long duration at various temperatures are required. Since this is an expensive procedure there has been a tendency to restrict flow tests to the narrow ranges of temperature of particular interest to those supporting the work or to otherwise curtail the work.

While charts shown in the accompanying Fig. 1 and Fig. 19 of the paper are useful in interpreting the flow test data it should not be forgotten that they are usually based on one melt or a limited number of test samples. The results from other lots, samples or melts of similar metals may differ by an amount equal to the differences between two different rates of flow on one lot of material. There is, therefore, still need for a study of the variations which may be expected industrially in the flow characteristics of given types of materials at different temperatures.

One point in the paper could be elaborated further to advantage. The authors apparently hold the view that the flow produced by stresses respectively above and well below their proportional limits are different in nature and specifically refer to the latter as "distortion" (page 103). Just what is meant is left somewhat in the dark.

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[1] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[2] U. S. Bureau of Standards *Technologic Paper No. 362.*

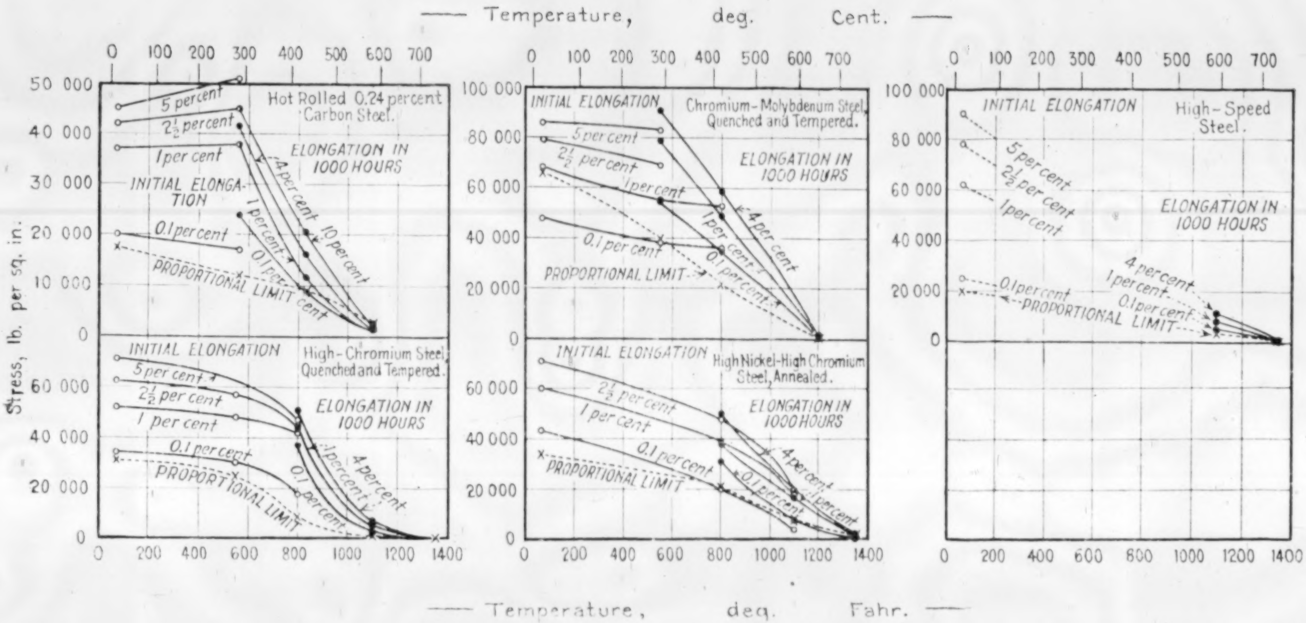

FIG. 1.—"Creep" or "Flow" Charts for Five Different Steels. These give the relations between stress temperature, elongation and time. Proportional limits were obtained in short time tension tests. For compositions and preliminary heat treatments of these steels see U. S. Bureau of Standards *Technologic Paper No. 362.*

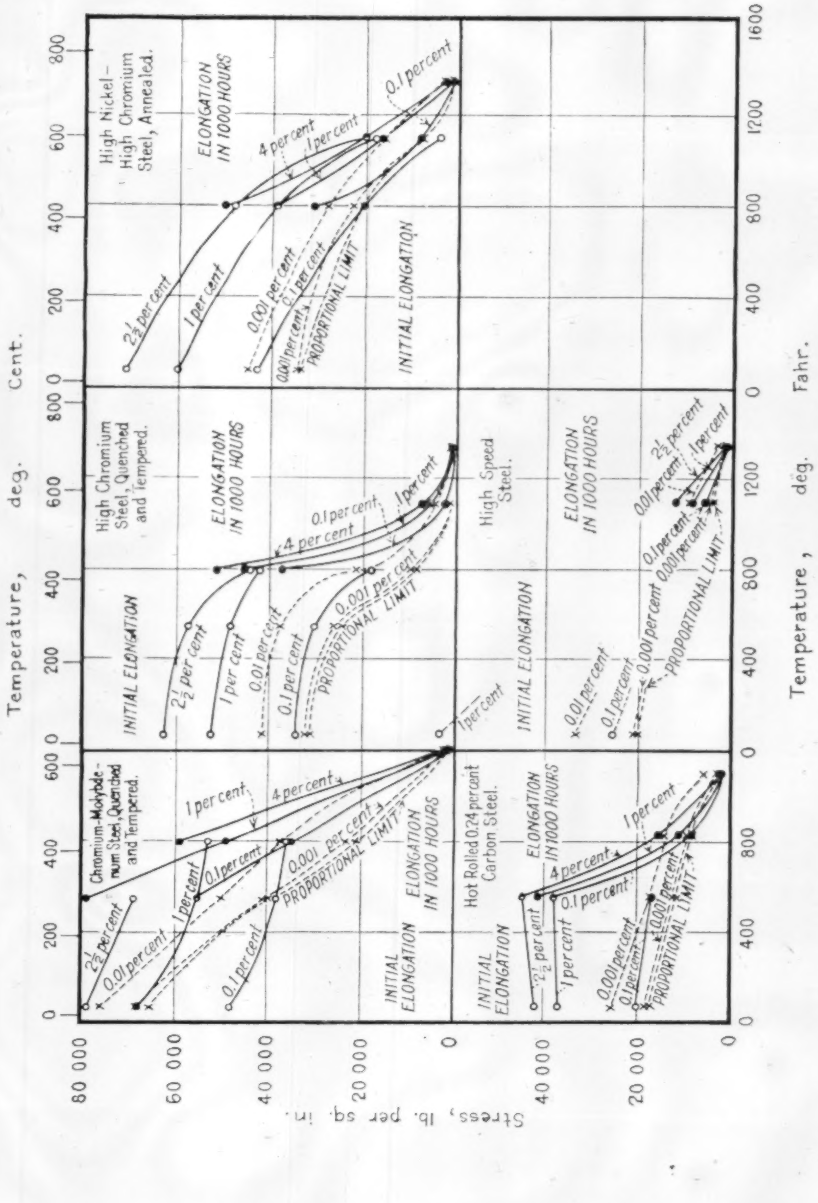

FIG. 2.—Correlation of Stress-Strain Relations in Tension Tests (dotted lines) with Creep Tests (full lines) at Corresponding Temperatures.

Mr. French.

Grain boundary material in polycrystallin metals is frequently a source of weakness at high temperatures, especially when the metal is subjected to stresses for long periods. It is possible that the flow under very low stresses, to which Messrs. Kanter and Spring refer, represents largely movement in the "disorganized" atoms at the grain boundaries whereas under relatively high stresses at the same temperatures movement also takes place within the crystals themselves.

Low rates of deformation should mean very long life, but if flow is concentrated in the grain boundary material, fracture may occur sooner than would be expected from extrapolating tests of several hundred hours' duration to periods of 10 years as is done in Fig. 19.

The writer can confirm the view that the proportional limits determined with accurate equipment are within the range of stresses producing low rates of flow, at least in the case of some steels. To better illustrate these relations, the accompanying Fig. 2 was prepared. The solid lines represent the creep charts in Fig. 1 and the dotted lines the stress-strain relations determined in short-time tension tests with an extensometer with smallest direct reading of 0.000004 in. The curves designated *P. L.* represent the smallest detectable deviation from the assumed proportionality between stress and strain; the remaining curves represent the deviations specified and illustrate what is in effect a change in sensitivity of the extensometers used.

It will be seen that the short-time test data reflect the changes found in the tedious flow tests although there are no quantitative relations between the two sets of curves. A deviation of 0.1 per cent from the assumed proportionality between stress and strain in the tension tests was generally observed at a stress which was higher than that producing similar deformation in 1000 hours in the flow tests at corresponding temperatures. This means that the short-time tests gave somewhat higher values than the long-time tests for a given accuracy of strain measurement. Furthermore, the differences became somewhat greater with increase in test temperature.

Mr. Duff.

MR. R. L. DUFF[1] (*presented in written form*).—Messrs. Kanter and Spring have struck a note in their paper which should invite considerable discussion concerning flow tests and interpretation of results. In many kinds of *equipment* any appreciable deformation is not permissible and stresses allowable over long periods of time at definite temperatures must be known in order that the apparatus may be properly designed. In the same *equipment*, however, stresses of known magnitude, but considerably greater than the long-time operating

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[1] Metallurgical Engineer, Standard Oil Development Co., Roselle, N. J.

Mr. Duff.

stresses may occur for short intervals of time, up to 15 minutes. Design must also take care of these stresses. These factors mean the procuring of data on long-time strengths and also short-time strengths.

The term creep stress, limiting creep stress, seems to be going through an evolution in regard to its real meaning. The load at a given temperature which will ultimately produce failure or produces persistent creep is a definition used considerably. The creep limit has also been defined as the stress necessary to produce a deformation of 1 per cent in 100,000 hours. Just how the data for the last definition is determined with any guarantee of its accuracy has yet to be explained to my satisfaction. Metal characteristics change when subjected to stresses and temperatures over long periods of time. Tests of 5000 hours duration tend to show that the rates of creep change, and that flow is, at least in some cases, a series of movements and stoppages, due to some internal changes.

Grain size effects as pointed out in the paper, open another field of investigation. Certainly extremely large grain size is undesirable, and it would seem that a small refined grain structure is not the best. From this arises the question as to the size of grain giving the greatest life at high temperatures commensurate with good steel making practice; also whether the same size grain holds for alloy steels as well as straight carbon steels.

The data presented by Messrs. Kanter and Spring with some data from other investigators bears out the slip interference theory, and it is hopeful that, with a little more data, a chemical analysis and careful microscopical examination may tell in a measure what the characteristics of a particular steel will be under high temperature.

The fact that carbon loses its potency as temperature increases, leads to the fact that alloy steels will be necessary in construction of pressure-temperature equipment if the present trend in temperatures and pressures continues. In many cases it is felt that certain parts of equipment are being subjected to maximum stresses in every-day operation, and slight temperature or pressure changes will cause deformation which renders the equipment useless or causes failure.

The amount of deformation that takes place due to the limiting creep stress is far greater, in a comparatively short time, than is allowable in most equipment, where 1 per cent deformation is a maximum figure. What must be determined is the stress at the working conditions, which, over the life of the equipment, will produce a deformation not over that allowed for the equipment.

I feel that so far the obtainable data has not shown that accurately determined proportional limits on short-time tests will give

Mr. Duff. safe design figures for apparatus to be used over long periods of time. Also, 500 to 1000 hours, from my own tests, seems a rather short time for any basic values.

Mr. Jasper. MR. T. MCLEAN JASPER[1] (*presented in written form*).—The question of testing at elevated temperatures is in great need of clarification. If I might state in specific terms what appears to be the most logical solution and invite discussion, it would be along the following lines.

The long-time ultimate strength at elevated temperatures is a value below which a load can be carried without failure for an indefinitely long time. It is believed that this value can be obtained by using several specimens of a steel, introducing them into the testing machine, keeping the temperature constant, at successively lower values of stress until a plot between the stress and time for failure for such specimen becomes parallel to the time ordinate.

This should show the load that can be carried indefinitely at a particular temperature for a particular steel and an appropriate factor of safety should be all that is necessary to use these data in design.

A year or so ago we read of the use of a very sensitive extensometer for the purpose of producing by a short-time elastic limit what was analogous to a long-time ultimate strength.[2] A study of Fig. 19 of the paper should convince the reader that such tests for evaluating the strength qualities of steel, to say the least, are very confusing. The writer thinks that they are of little value to use as a criterion of stabilized values for design. The writer has made tests which have checked up the general findings represented above.

At ordinary temperatures the values of the long-time ultimate strength and the short-time yield point are not identical. Neither are they at elevated temperatures.

The test results in the paper represent a considerable amount of work which should be very much appreciated. It is believed that creep tests at low values of stress and at definite temperatures taking five or six months will show much different results from those represented in this paper. The rate of creep will become less as the time is increased up to a certain point in stress. Above this value in stress the creep will progressively increase in value.

Vessels in continuous service for over two years at 875° F. (470° C.) and under 6000 lb. per sq. in. operating stress at this temperature show that creep is negligible for the 20 to 30-per-cent carbon steel. Long-time ultimate strength tests at 900° F. (480° C.) show this metal to have a strength value of 18,000 lb. per sq. in. It is believed, there-

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[1] Director of Research, A. O. Smith Corporation, Milwaukee, Wis.

[2] H. J. French, "Methods of Test in Relation to Flow in Steels at Various Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 7 (1926).

Mr. Jasper. fore, that the method of testing that will give the long-time ultimate strength should be recommended by the Society with an appropriate factor of safety. The creep tests are very valuable in arriving at an appropriate factor of safety to use. These tests are very time consuming and it is believed can be largely discontinued except in a few special laboratories.

After testing steel by long-time methods, the writer has adopted the working load curve shown in the accompanying Fig. 3 for pressure vessels made of 20 to 30-per-cent carbon steel when used at high temperatures.

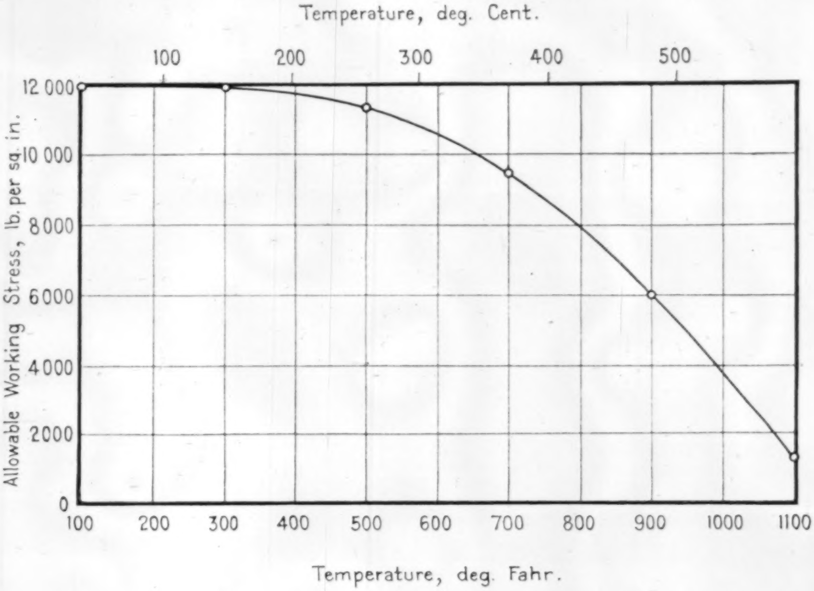

FIG. 3.—Curve Showing Allowable Working Stress for 0.20 to 0.36-per-cent Carbon Steel at Various Temperatures.

Mr. Templin. MR. R. L. TEMPLIN.[1]—While the authors of this paper have pointed out that there are a number of factors that affect the results in tests of this character, there is one that should be emphasized as a result of tests we have made at room temperature similar to those reported. I refer to the necessity for testing equipment of the kind indicated to be free from vibration during the test. We have found that test results will vary over a wide range, depending on the amount of vibration that exists in the testing machines during the test. I should like to ask the authors if they have given consideration to

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

Mr. Templin. that factor in their test work, and if any provisions have been made to take account of variations that may occur due to that trouble?

Mr. French. MR. FRENCH.—I should like to ask Mr. Templin whether he refers to aluminum alloys or ferrous alloys in his statement?

Mr. Templin. MR. TEMPLIN.—I referred to tests on magnesium alloys and aluminum alloys.

Mr. McVetty. MR. P. G. MCVETTY[1] *(by letter)*.—The authors of this paper have made a valuable contribution to the literature of long-time tension tests.

On account of the large amount of data presented, the necessity of restricting the discussion of test methods is obvious. There are a few points, however, on which additional information would be desirable.

In Fig. 6 of the paper, proportional limit values are given for several materials at different temperatures. It is a well-known fact that measured values of proportional limit are influenced by the sensitivity of the extensometer used.[2] A statement concerning the extensometer used would be of interest. This information would aid also in the interpretation of Fig. 19, in which agreement is shown in the temperature range of 900 to 1200° F. (480 to 650° C.) between the short-time proportional limit and the stress, causing one per cent of flow in 1000 hours.

Referring to the matter of temperature control, the use of a control thermocouple in contact with the furnace winding may cause some difficulty. Fluctuations in specimen temperature result from two main causes, namely, variations in line voltage and variations in room temperature. There is, of necessity, a considerable temperature gradient between the furnace winding and the specimen or grip adjacent to it. If the room temperature remains constant, control is best effected by control of winding temperature. If, however, the room temperature varies, the winding temperature must be changed accordingly to maintain constant specimen temperature. It appears that the method used is advantageous only if tests are conducted in a constant temperature room. Otherwise, any change in room temperature requires a corresponding hand adjustment of furnace current.

The use of an extensometer which shows extension on one side of the specimen only makes it difficult to separate initial and secondary creep. It gives no opportunity to detect faulty alignment in the machine or lack of homogeneity of the material in the specimen.

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[1] Mechanical Engineer, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] P. G. McVetty and N. L. Mochel, "Tensile Properties of Stainless Iron and Other Alloys at Elevated Temperatures," *Transactions*, Am. Soc. Steel Treating, Vol. XI, p. 88 (1927).

Mr. McVetty

The importance of this factor in short-time tension tests has been previously shown.[1] It is evident that any eccentricity of loading will affect the measurement on one side of the specimen. The amount and direction of this effect will depend upon the angular relation between the line of measurement and the direction of bending. The accurate evaluation of the effect of eccentricity requires measurement in at least three places around the specimen, but two opposite measurements are usually considered sufficient.

Messrs. Kanter and Spring.

MESSRS. J. J. KANTER[2] and L. W. SPRING[3] (*authors' closure by letter*).—The comments made in the preceding discussions in reference to the methods, data and interpretations given in this paper are for the most part substantiative of the authors' viewpoint and are valuable for the attention they call to certain details and considerations, which, for the sake of brevity, were perhaps not sufficiently emphasized.

Regarding Mr. French's criticism, by "distortion" as here used, is meant the flow which may occur in equipment under service stresses, which from the various evidence does not seem to cause serious change in the properties of the steel itself. That there is any qualitative difference of flow characteristics with respect to the side of the proportional limit on which the stress falls, would be a view quite untenable with the data presented. That there are still many controversial aspects to the explanation of flow or creep curves in terms of rearrangements in the crystallin aggregate is fully appreciated, and as Mr. French points out, it is possible that in this regard magnitude of stress may be a controlling factor over the ratio between grain boundary movements and slip within the crystals themselves.

The authors are in accord with Mr. Duff's view that far-reaching extrapolations upon the basis of tests not more than 1000 hours in duration may not be accurate prophecy. As was stated in presenting the paper in question, some of the tests incorporated in the data given are being permitted to run for much longer periods in order to check some of the assumptions made. It seems almost needless to add that tests enough to render possible assumptions as to the behavior of materials in flow require extensive equipment when run for only a few hundred hours, and comprehensive tests of longer duration cannot be forthcoming until longer time has elapsed. It would indeed be fortunate if, as Mr. Duff suggests, high temperature characteristics of steel could be predicted from chemical analysis and careful microscopical examination.

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[1] P. G. McVetty, Discussion of paper by M. F. Sayre on "A New Type of Mirror Extensometer," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 666 (1926).

[2] Testing Engineer, Crane Co., Chicago, Ill.

[3] Chief Chemist and Metallurgist, Crane Co., Chicago, Ill.

Messrs. Kanter and Spring.

Mr. Jasper's statement that the rate of creep will become less as the time is increased up to a certain point in stress, may perhaps be true at temperatures not exceeding 800° F. (425° C.) in the case of steel. However, at temperatures of 900 and 1000° F. (480 and 540° C.), tests by the authors of over 5000 hours duration at low values of stress show no tendency to decrease in rate of creep after about the first 1000 hours of testing. Although longer testing may show somewhat different values for temperatures below 800° F. (425° C.) the indications are that in the higher range it would not. How Mr. Jasper's "strength value" of 18,000 lb. per sq. in. at 900° F. (480° C.) is arrived at and the significance of the safety factor of 3, are not entirely clear. Values obtained by the method of plotting successively lower values of stress until the stress-time curves become parallel to time ordinate depend to a great extent upon the scale used in plotting and an entirely different looking picture is obtained where a logarithmic time scale is used.

In answer to the question raised by Mr. Templin relative to effect of vibrations on the authors' flow tests, the first five units built were placed upon shock absorbers (as described on page 84). Little difference is noticed in rate of flow produced under a given load whether the shock absorbers are in action or not. While they, no doubt, exert some influence upon results, vibrations do not seem to be a considerable factor for the steels tested.

Referring to Mr. McVetty's comment, the extensometer used attaches directly to the 2-in. breaking section of the test bar, with comparison rods extending beneath the furnace. Extensions are read on two dials actuated from opposite sides of the specimen and calibrated to read directly to 0.0001 in. He points out the possibility of temperature fluctuations in long-time tests controlled through thermocouples in contact with furnace winding, due to room temperature variations. Some difficulty has been encountered in this regard, but the variations have not been found of great enough magnitude to be serious. Corrections for such are made through hand adjustment. Very rarely have these variations in temperature exceeded ±5° F. (3° C.) as indicated by the multiple Leeds & Northrup recorder.

While it is appreciated that for refinement in long-time tests, extension measurements must be made on opposite sides of the specimen to obtain true flow-time curves, the authors have taken precaution against exaggerated eccentricity of loading. Measurement of the immediate or elastic extension upon load application is taken in each test and checked for modulus of elasticity. In case there is eccentricity, an unreasonable modulus value is shown, and the alignment is then changed to alter the condition.

SOME FACTORS INVOLVED IN CORROSION AND CORROSION-FATIGUE OF METALS[1]

By D. J. McAdam, Jr.[2]

Synopsis

Part I of this paper gives a brief resumé of previous work, an outline of continued investigation, and a description of material and methods.

Parts II and III discuss the interaction of cyclic stress-range, number of cycles, and time, in causing penetration of steel under corrosion. A variety of carbon and alloy steels, including stainless iron, were used. Effects of three kinds of water are discussed. The investigation was carried out by "prior-corrosion fatigue" tests in which the specimens were first subjected to simultaneous corrosion and cyclic stress below the corrosion-fatigue limit, and then were subjected to fatigue test in air. The fatigue limit obtained in the second stage is used as a criterion of the effect of corrosion. Results of these fatigue tests are presented in ordinary stress-cycle graphs. From the fatigue limits obtained from these Type 1 graphs three other types of graphs are derived to illustrate the effect of cyclic stress-range, time, and number of cycles.

Part II discusses the effect of varying cyclic stress range, with constant time and number of cycles. Among the subjects discussed are notching limit, notch-depth limitation, corrosion-fatigue limit, stress-concentration ratio, and effect of electrochemical and physical properties on the behavior of metals under simultaneous corrosion and cyclic stress.

Part III discusses the effect of time and number of cycles on penetration of metal under corrosion. The effect of cyclic stress-range is still kept in view and its interaction with the other two variables is discussed. Among the subjects discussed are notch-depth limitation, penetration per cycle, time rate of penetration, and effect of time and number of cycles on the notching limit.

Part IV discusses effect of diameter or thickness of specimen on the corrosion-fatigue limit. Results obtained with specimens 2.3 in. in diameter are compared with results obtained with tapered specimens previously described.

Part V discusses torsional corrosion-fatigue. Corrosion-fatigue limits obtained in this way are compared with limits obtained by rotating cantilever method. Photomicrographs of corrosion-fatigue cracks are presented.

PART I.—OUTLINE OF INVESTIGATION, DESCRIPTION OF MATERIAL AND METHODS

Previous Investigation of Corrosion-Fatigue and Prior-Corrosion Fatigue:

Results of investigation of corrosion-fatigue of metals at the U. S. Naval Engineering Experiment Station, Annapolis, Md., have been presented by the author in several papers (1, 2, 3, 4).[1] In these papers, references were also given to the work of Haigh in 1917(5). The investigation at Annapolis included steels, and alloys of copper, of nickel, and of aluminum. The experiments showed that even slight corrosion simultaneous with fatigue may cause failure at nominal stresses far below the ordinary endurance limit. It was also shown that, for most metals, severe stressless corrosion prior to fatigue is much less damaging than even slight corrosion simultaneous with fatigue. For some aluminum alloys, however, as shown in a recent paper(4), corrosion prior to fatigue is about as damaging as corrosion simultaneous with fatigue. The term "corrosion-fatigue" has been restricted to mean the simultaneous action of corrosion and fatigue. The action of corrosion followed by fatigue has been called "prior-corrosion fatigue."

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[1] Published by permission of the Secretary of the Navy.

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

A stress-cycle graph representing corrosion-fatigue is a curve approaching a horizontal asymptote, which may be called a "corrosion-fatigue limit." The term "endurance limit" has been restricted to mean the fatigue limit obtained by tests in air with specimens as free as possible from stress concentration and from corrosion.

As a result of a study of the relationship between corrosion-fatigue and prior-corrosion fatigue a tentative analysis of the corrosion-fatigue process was given by the author in a recent paper(4). According to this analysis, if the initial stress is below the endurance limit but above the corrosion-fatigue limit, the corrosion-fatigue process is divided into two stages. The first stage is the formation of pits. As the initial stress is below the endurance limit, such pit formation is assumed to be due to electrolytic solution pressure, not to fatigue. The electrolytic solution pressure that causes the pitting, however, is not the solution pressure of stressless corrosion, but is possibly an enhanced solution pressure due to the cyclic stress. Whatever may be the reason, a metal in the first stage of corrosion-fatigue behaves as if its solution pressure were increased by the cyclic stress.

As the pit progresses, both the actual stress and the effective solution pressure are increased. The increase of actual stress is due to the usual stress concentration at an abrupt change of section. The increase of solution pressure is due to the increase of stress.

The mutual intensification of stress and solution pressure continues until the actual stress at the bottom of the pit surpasses the endurance limit of the metal. The second stage of corrosion-fatigue then begins. This second stage is merely fatigue accelerated by corrosion. In this stage the crack advances at a continually accelerated rate until the specimen breaks.

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[1] The boldface numbers in parentheses refer to the papers given in the list of reference appended hereto.

If the initial stress is above the endurance limit, the first stage of corrosion-fatigue is absent and the process is merely fatigue accelerated by corrosion. The absence of the first stage accounts, in part at least, for the previously described(3) abrupt bending to the left as the corrosion-fatigue graph rises above the endurance limit.

Previous Investigation of Effect of Cyclic Stress on Corrosion:

If the above-described analysis of the corrosion-fatigue process is correct, the depth and sharpness of the pits formed in the first stage depend not only on the natural susceptibility of the alloy to corrosion pitting (the electrolytic potential is usually an index of this susceptibility), but also on the influence of cyclic stress on corrosion pitting. Knowledge of the effect of cyclic stress on corrosion pitting, therefore, is necessary for an understanding of corrosion-fatigue. As little was known about the effect of cyclic stress on corrosion, an investigation of this subject was started at the Naval Engineering Experiment Station.

Some results of the investigation were given in a recent paper(6). These results show that the above-described analysis of the corrosion-fatigue process is essentially correct. The results not only throw light on the corrosion-fatigue process, but are applicable to all conditions of service in which metals or alloys are subjected to simultaneous corrosion and cyclic stress, whether the stress be below or above the corrosion-fatigue limit. As this paper will be referred to oftener than any other paper it will be called merely "the previous paper."

The previous paper(6) discussed the effect of cyclic stress-range on corrosion of a number of ferrous and non-ferrous metals, and presented the results of a preliminary investigation of the effect of time and number of cycles on corrosion of steel. The results obtained with a chromium-vanadium steel showed a complex relationship between cyclic stress-range and depth of corrosion pitting. This relationship seemed of enough theoretical and practical importance to justify a thorough investigation of the effect of cyclic stress-range, time, and number of cycles on corrosion of a variety of carbon and alloy steels. Results of this investigation are included in Parts II and III of the present paper.

Outline of Continued Investigation of Effect of Time and Cyclic Stress on Corrosion:

The factors involved in corrosion of metals may be classified in at least seven groups: (1) Nature of corroding agent; (2) temperature; (3) time; (4) cyclic stress range; (5) number of cycles; (6) corrosion resistance of the alloy; and (7) physical properties of the alloy.

In the previous paper(6), only one kind of corroding agent (carbonate water) was used, and the temperature was kept within as narrow a range as was practicable. The influence of factors (1) and (2), therefore, was not included in the previous investigation. In the present investigation, however, three kinds of water were used, but the temperature was kept within as narrow a range as was practicable. The present investigation, therefore, includes the effect of cyclic stress range, number of cycles, time, and kind of corroding agent.

The effect of corrosion that is given chief consideration in the present paper, as in the previous paper, is the effect on the fatigue limit. This effect is determined by comparing the fatigue limits of previously corroded specimens with the endurance limit of the metal. For this purpose, specimens have been subjected to corrosion at various cyclic stresses from zero to the corrosion-fatigue limit, for various times, and at various cycle frequencies. The specimens have then been oiled and subjected to fatigue in air.

Each experiment, therefore, consisted of two stages, a pitting stage and a fatigue stage. In such experiments, which may be called "prior-corrosion fatigue" tests, the nominal stress in the pitting stage is usually lower than in the fatigue stage. In this respect a prior-corrosion fatigue test differs from a corrosion-fatigue test.

In these experiments, as in those described in previous papers(1,2,3,4,6), the specimens were corroded in a water stream, which was diagonally applied so as to surround with water the tapered portion of the specimen. In the experiments described in the present paper, a soft water was used in addition to the two kinds of water that were previously used, a carbonate water and Severn River water. The soft water is an artesian well water that is softened by chemical treatment. The composition of the three kinds of water is given in Table IV.

During the winter months, the water before reaching the specimen was artificially heated to a temperature between 65 and 75° F., though on some of the coldest nights the temperature occasionally dropped as low as 60° F.

Outline of Investigation of Effect of Size of Specimen on Corrosion-Fatigue:

In previous experiments by the author on corrosion-fatigue and on the effect of time and cyclic stress on corrosion, the test specimens used were tapered specimens, the largest diameter of the tapered portion being ½ in. It is obviously important to know the effect of corrosion-fatigue on specimens of larger and smaller section. Part IV discusses from a theoretical viewpoint the possible effect of decrease of section on the corrosion-fatigue limit and presents data showing the effect of increase of diameter to 2.3 in.

Outline of Investigation of Torsional Corrosion-Fatigue:

In the experiments on corrosion-fatigue by the author, described in previous papers(1,2,3,4,6), rotating-cantilever specimens have been used. It is of theoretical and practical importance to know the relation between the corrosion-fatigue limits obtained by this method and the limits obtained under torsional corrosion-fatigue. Part V of the present paper describes an investigation of this subject.

Machines and Specimens:

The rotating-cantilever machines and specimens have been described in previous papers(7,8). The conically tapered specimen is so designed that the maximum stress is ¾ in. out from the inner fillet, and the stress varies only about 1.5 per cent over a length of 1.5 in. With this specimen, therefore, a comparatively large region may be subjected to simultaneous corrosion and cyclic stress.

The alternating-torsion machines used in investigation of torsional corrosion-fatigue are the same that were used in a previous investigation of the "endurance range" of steel(10).

The method of alternate longitudinal and transverse polishing of specimens has been previously described(8,9). The surface finish by this method is sufficiently smooth to permit examination of the structure at a magnification of 100. Details of machining and polishing are tabulated and kept on file at the Naval Engineering Experiment Station. This information is available for those who are interested.

Material:

Material for this investigation was generously supplied by the American Rolling Mill Co., the Firth-Stirling Steel Co., the International Nickel Co., and the Aluminum Co. of America. Specimens of chrome-nickel steel 2½ in. in diameter were generously supplied by

TABLE I.—CHEMICAL COMPOSITION OF MATERIAL.

All Values are Averages of at Least Two Determinations, Usually More

Material	Designation	Carbon, per cent	Manganese, per cent	Phosphorus, per cent	Sulfur, per cent	Silicon, per cent	Nickel, per cent	Chromium, per cent	Vanadium, per cent	Molybdenum, per cent	Copper, per cent	Aluminum, per cent	Iron, per cent	Magnesium, per cent
Ingot Iron	IZ	0.018	0.016	0.005	0.031	0.002	.....	.....	.....	.....	.....	.....	.....	.....
0.09-per-cent Carbon Steel	IY	0.09	0.48	0.114	0.107	0.01	.....	.....	.....	.....	.....	.....	.....	.....
0.24-per-cent Carbon Steel	IH	0.24	0.57	0.005	0.036	0.006	.....	.....	.....	.....	.....	.....	.....	.....
0.29-per-cent Carbon Steel	IX	0.29	0.54	0.017	0.041	0.01	.....	.....	.....	.....	.....	.....	.....	.....
0.56-per-cent Carbon Steel	JA	0.56	0.67	0.008	0.030	0.12	.....	.....	.....	.....	.....	.....	.....	.....
High-Carbon Steel	EM	1.09	0.33	0.023	0.015	0.28	.....	.....	.....	.....	.....	.....	.....	.....
Chromium-Molybdenum Steel	AL	0.33	0.71	0.005	0.017	0.13	.....	0.93	.....	0.15	.....	.....	.....	.....
Chromlum-Vanadium Steel	AZ	0.46	0.69	0.007	0.010	0.24	.....	0.88	0.14	.....	.....	.....	.....	.....
Chromium-Vanadium Steel	AR	0.50	0.70	0.013	0.012	0.25	.....	0.99	0.16	.....	.....	.....	.....	.....
Nickel Steel	IW	0.28	0.50	0.15	0.36	0.19	3.70	0.26	.....	.....	.....	.....	.....	.....
Chromium-Nickel Steel	BC	0.28	0.69	0.014	0.016	0.21	1.51	0.73	.....	.....	.....	.....	.....	.....
Chromium-Nickel Steel	AH	0.39	0.69	0.015	0.023	0.17	1.67	0.82	.....	.....	.....	.....	.....	.....
Stainless Iron	IB	0.10	0.34	0.007	0.030	0.29	0.51	13.30	.....	.....	.....	.....	.....	.....
Monel Metal, Cold-Drawn	IU	0.090	1.00	0.013	0.026	0.009	71.30[a]	.....	.....	.....	25.72	.....	1.83	
Aluminum, Hard-Temper	IMB	.....	0.003	.....	.....	0.13	.....	.....	.....	.....	0.09	99.47[a]	0.31	None
Aluminum-Manganese Alloy, Hard-Temper	ILB	.....	1.33	.....	.....	0.28	.....	.....	.....	.....	0.10	97.69[a]	0.60	None
Duralumin, Tempered	GT	.....	0.50	.....	.....	1.02	.....	.....	.....	.....	3.72	94.06[a]	0.42	0.28

[a] Obtained by difference.

TABLE II.—HEAT TREATMENTS OF MATERIAL.

Material	Designation	Heated to, deg. Fahr.	Time Held, minutes	Cooled in	Reheated to, deg. Fahr.	Time Held, minutes	Cooled in
Ingot Iron	IZ-17.5	1750	45	Furnace			
0.09-per-cent Carbon Steel	IY	As received					
0.24-per-cent Carbon Steel	IH-W-9	1650	60	Water	900	120	Air
0.24-per-cent Carbon Steel	IH-16.5	1650	60	Furnace			
0.29-per-cent Carbon Steel	IX-W-9	1650	60	Water	900	120	Air
0.56-per-cent Carbon Steel	JA-W-11.75	1500	60	Water	1175	60	Furnace
0.56-per-cent Carbon Steel	JA-15	1500	60	Furnace			
High-Carbon Steel	EM-14.75	1475	60	Furnace			
Chromium-Molybdenum Steel	AL-O-10	1575[a]	60	Oil	1000	60	Furnace
Chromium-Vanadium Steel	AZ-O-7.5	1575[a]	60	Oil	750	60	Furnace
Chromium-Vanadium Steel	AZ-O-10	1575[a]	60	Oil	1000	60	Furnace
Chromium-Vanadium Steel	AZ-15.75	1575[a]	60	Furnace			
Chromium-Vanadium Steel	AR-O-10	1575[a]	60	Oil	1000	60	Furnace
Nickel Steel	IW-W-10	1450[b]	60	Water	1000	120	Furnace
Chromium-Nickel Steel	BC-W-10	1550	60	Water	1000	120	Furnace
Chromium-Nickel Steel	AH	As received[c]					
Stainless Iron	IB-W-10	1800	60	Water	1000	120	Air
Stainless Iron	IB-W-11	1800	60	Water	1100	120	Air
Stainless Iron	IB-W-12	1800	60	Water	1200	120	Air
Stainless Iron	IB-16	1600	120	Furnace			
Monel Metal, Cold Drawn	IU-8	800	180	Furnace			
Aluminum, Hard Temper	IMB	As received[d]					
Aluminum-Manganese Alloy, Hard Temper	ILB	As received[d]					
Duralumin, Tempered	GT	As received[d]					

[a] Previously heated to 1700 deg. Fahr., held 60 minutes, and cooled in air.
[b] Previously heated to 1675 deg. Fahr., held 60 minutes, and cooled in air.
[c] Heat-treated airplane crankshaft material.
[d] Heat-treated by manufacturer.

TABLE III.—RESULTS FROM TENSION AND CHARPY TESTS RESULTS; AVERAGES AND MEAN DEPARTURES FROM AVERAGES.

Unless otherwise indicated, each value is the average of two determinations.

Material	Designation	Condition	Tensile Strength, lb. per sq. in.	Johnson's Limit, lb. per sq. in.	Proof Stress, lb. per sq. in.	Elastic Limit, lb. per sq. in.	Proportional Limit, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Charpy Impact Values, ft-lb.
Ingot Iron	IZ-17.5	Annealed	42 600 ±300	15 000	15 000	10 800 ±300	10 500 ±500	47.0	69.0	10.4 ±0.9
0.09-per-cent Carbon Steel	IY	As Received	73 900 ±100	55 500 ±1 500	56 500 ±500	51 000	35 000	21.5 ±0.5	44.7 ±0.5	3.5 ±0.2
0.24-per-cent Carbon Steel	IH-W-9	Quenched and Drawn	78 700^c ±3 400	55 700^c ±5 100	54 300^c ±5 600	53 700^c ±5 800	50 000^c ±3 300	26.2^c ±1.4	68.1^c ±0.9	42.8^c ±0.4
0.24-per-cent Carbon Steel	IH-16.5	Annealed	55 600^c ±1 500	33 300^c ±4 100	34 000^c ±4 000	28 000^c	24 700^c ±1 100	39.0^c ±1.3	59.7^c ±1.8	12.1^c ±2.2
0.29-per-cent Carbon Steel	IX-W-9	Quenched and Drawn	81 800 ±2 200	55 000 ±1 000	56 000	53 500 ±2 500	47 500 ±2 500	25.3 ±0.8	64.6 ±2.1	42.3^c ±0.7
0.56-per-cent Carbon Steel	JA-W-11.75	Quenched and Drawn	112 300^c ±3 800	80 700^{a,c} ±3 800	....	....	79 700^c ±3 800	23.3^c ±1.3	58.5^c ±1.7	4.0^c ±0.2
0.56-per-cent Carbon Steel	JA-15	Annealed	90 900^c ±1 700	40 700^{a,c} ±1 100	....	....	39 700^c ±1 100	22.7^c ±1.4	29.8^c ±1.6	4.5^c ±0.1
High-Carbon Steel	EM-14.75	Annealed	103 400 ±4 100	60 300 ±2 800	60 600 ±2 900	55 800 ±3 700	49 500 ±2 300	29.5 ±1.3	50.8 ±2.4	12.9^c ±1.3
Chromium-Molybdenum Steel	AL-O-10	Quenched and Drawn	117 600^f ±2 200	90 700^f ±1 600	91 500^f ±1 800	88 000^f ±1 900	81 000^f ±2 900	20.6^f ±1.1	65.1^f ±0.7	39.6 ±0.4
Chromium-Vanadium Steel	AZ-O-7.5	Quenched and Drawn	179 300^c ±4 600	128 600^c ±5 800	129 200^c ±5 300	110 000^c ±10 000	93 300^c ±4 400	14.0^c ±1.0	47.8^c ±3.9	7.6^c ±2.0
Chromium-Vanadium Steel	AZ-O-10	Quenched and Drawn	146 800^g ±7 200	120 100^g ±4 700	121 000^g ±3 500	109 900^g ±5 200	83 700^g ±7 300	16.6^g ±1.0	54.4^g ±1.4	17.7 ±1.6
Chromium-Vanadium Steel	AZ-15.75	Annealed	97 800^d ±1 200	56 200^d ±1 400	57 100^d ±1 400	50 000^d ±2 500	42 100^d ±2 700	28.1^d ±1.6	53.8^d ±2.1	17.1 ±1.0
Chromium-Vanadium Steel	AR-O-10	Quenched and Drawn	182 200 ±1 600	150 500 ±5 500	149 000 ±9 000	135 000 ±10 000	87 500 ±2 500	14.5 ±1.5	49.7	19.3 ±1.3
Nickel Steel	IW-W-10	Quenched and Drawn	130 400^c ±2 800	118 000^c ±3 300	118 700^c ±2 900	115 000^c ±3 300	90 700^c ±900	19.3^c ±0.8	61.3^c ±0.4	29.0 ....
Chromium-Nickel Steel	BC-W-10	Quenched and Drawn	138 250^c ±3 300	122 000^c ±5 300	123 000^c ±5 300	119 700^c ±5 100	96 700^c ±4 400	19.3^c ±1.4	57.7^c ±1.4	30.6 ±3.0
Chromium-Nickel Steel	AH	As Received	134 800^d ±1 600	110 400^d ±3 400	112 500 ±4 500	103 800 ±3 700	87 500^d ±6 200	18.4^d ±0.4	57.8^d ±1.6	25.8 ±1.1
Stainless Iron	IB-W-10	Quenched and Drawn	175 800^d ±3 400	102 600^d ±1 900	96 500^d ±4 000	79 500^d ±3 000	72 500^d ±2 500	18.8^d ±1.3	61.2^d ±3.2	6.7^c ±0.9
Stainless Iron	IB-W-11	Quenched and Drawn	124 700^c ±800	91 000^c ±1 300	90 700^c ±400	84 000^c ±4 000	69 700^c ±400	22.0^c ±1.3	70.1^c ±0.6	45.0^b ....
Stainless Iron	IB-W-12	Quenched and Drawn	104 700 ±3 200	72 500 ±2 500	76 000 ±3 000	68 500 ±3 500	59 000 ±3 000	23.0 ±1.0	70.0	47.6 ±2.6
Stainless Iron	IB-16	Annealed	80 600^c ±400	34 000^c ±700	32 700^c ±900	21 300^c ±1 100	20 000^c	34.5^c ±1.3	73.5^c	45.5 ±3.0
Monel Metal, Cold-Drawn	IU-8	Low Anneal	83 000 ±500	52 500	52 800 ±300	44 000 ±1 000	40 000	36.5 ±0.5	70.5	63.0 ±1.8
Aluminum, Hard Temper	IMB	As Received	20 800 ±200	15 000	15 000	12 500 ±500	10 500 ±500	15.8 ±0.8	63.0 ±0.5	16.1 ±0.6
Aluminum-Manganese Alloy, Hard Temper	ILB	As Received	27 500 ±400	20 000	16 500 ±500	11 500 ±500	10 000	11.0 ±0.5	39.3 ±0.3	9.1 ....
Duralumin, Tempered	GT	As Received	69 100^e ±900	45 300^e ±1 100	45 500^e ±1 100	40 700^e ±900	36 800^e ±600	17.8^e ±1.2	22.8^e ±2.2	10.1^d ±0.7

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[a] Yield point. [b] One determination only.

[c] Average of three determinations. [d] Average of four determinations.

[e] Average of six determinations. [f] Average of ten determinations.

[g] Average of twelve determinations.

the Wyman Gordon Co. All the other alloys used were supplied in the form of round bars 1 in. in diameter.

Chemical Composition and Heat Treatment.—The chemical composition of the steels used is given in Table I. Details of heat-treatment are given in Table II.

Tension and Impact Tests:

Results of tension and Charpy impact tests are given in Table III. In this table, "elastic limit" means the highest stress that leaves no appreciable permanent deformation after removal of the load. "Proof stress" means the stress that results in a permanent deformation of 0.0001 in. per inch of length after removal of the load.

TABLE IV.—WATER ANALYSES.

Results Expressed in Parts Per Million.

	Carbonate Water	Soft Water	Severn River Water
Silica	11.66	12.00	....
Iron and aluminum oxides	......	1.37	....
Calcium sulfate	1.88	13.71	....
Calcium carbonate	199.85	.....	....
Magnesium chloride	17.48	.....	....
Sodium chloride	142.26	8.57	....
Sodium sulfate	......	26.22	....
Total solids	373.13	61.87	....
Chlorine, yearly maximum	......	.....	7200
Chlorine, yearly minimum	......	.....	1700
Chlorine, yearly average	......	.....	4500
Chlorine, recent maximum	......	.....	3800
Chlorine, recent minimum	......	.....	2400
Chlorine, recent average	......	.....	3500

NOTE.—The chlorine percentages designated as "recent" represent the condition of the Severn River water at the time of the experiments described in the paper.

As the chlorine content of sea water is about 2 per cent, the salinity of the Severn River water ranges from about 8 to 36 per cent of the salinity of sea water. During the experiments described in the paper the salinity was about one-sixth that of sea water.

PART II.—EFFECT OF CYCLIC STRESS-RANGE ON CORROSION.

General Description of Stress-Cycle Graphs:

In Figs. 1 to 6 are shown by stress-cycle graphs results of prior-corrosion fatigue tests of a number of steels. In these tests specimens of various steels were corroded at various cyclic stress-ranges and cycle frequencies, for various numbers of cycles, and in three kinds of water. They were then oiled and subjected to fatigue test at 1450 r.p.m. in air. In future discussion, the corrosion stage will be called the "first stage" or "stage 1." The fatigue stage of the prior-corrosion fatigue test will be called the "second stage" or "stage 2."

Each group of graphs in Figs. 1 to 6 has its own logarithmic scale of abscissas, which is indicated by figures along the graph.

The small inverted triangles, squares, etc., with adjacent letters, represent results of prior-corrosion fatigue tests. For Figs. 1 to 5 the meanings of these marks are indicated in a general legend. In Fig. 6 the meanings of the marks are indicated by a special legend in the figure. In Figs. 1 to 6 the kind of mark indicates the kind of water used, the cycle frequency and the number of cycles applied in the first stage. The position of each of these marks on the graph represents the result of the second or fatigue stage of a prior-corrosion fatigue test.

Each zigzag line shown in these figures represents the course of a prior-corrosion fatigue test in which the stress was raised at intervals until the specimen failed. The lower end of the zigzag line represents the original stress and the number of cycles endured at this stress. The course of the line represents the stress increases and the cycle intervals at which these increases were made.

Each light solid line in Figs. 1 to 6 represents the stress-cycle relationship as obtained by stage 2 tests after corrosion in stage 1 at the stress, cycle frequency and number of cycles indicated. The numbers at the right of some of these lines indicate the stage 1 stress. Results of numerous experiments, presented in this paper and in the previous paper(6), have shown that the lower part of each of these lines is practically parallel to the corresponding part of the endurance graph for uncorroded specimens. For this reason the position of each of the lines can usually be determined satisfactorily by test of only one or two specimens.

For comparison, corrosion-fatigue graphs and ordinary endurance graphs for most of the steels have also been included. The small triangles with vertex upward represent results of corrosion-fatigue tests. To avoid obscuring results of prior-corrosion fatigue tests, individual results of corrosion-fatigue and ordinary endurance tests have sometimes been omitted and the results have been represented only by stress-cycle lines. Fatigue of uncorroded specimens is represented by heavy solid lines and corrosion-fatigue is represented by broken lines.

Effect of Cyclic Stress-Range as Illustrated by Stress-Cycle Graphs:

To investigate the effect of any one of the three factors, cyclic stress, number of cycles, and time, one of the factors must be varied while the other two are kept as nearly constant as possible. To investigate the effect of cyclic stress on corrosion of steel, experiments have been made with various cyclic stresses, at 1450 r.p.m., and for 20 million cycles. This number of cycles was selected because beyond 20 million cycles the corrosion-fatigue graph for steels is nearly horizontal.

Results of these experiments are represented in Figs. 1 to 5 by groups of graphs, each group illustrating the effect of varying cyclic stress-range on the relative position of the graphs. In the same figures are shown other groups of graphs illustrating the effect of cyclic stress at other frequencies and for other numbers of cycles; these will be discussed in Part III.

EFFECT OF CYCLIC STRESS-RANGE ON CORROSION OF STEELS IN CARBONATE WATER

Effect as Illustrated by Stress-Cycle Graphs:

Effect of cyclic stress-range on corrosion of carbon and high-strength alloy steels in carbonate water is shown in Figs. 1 to 4. By "high-strength alloy steels" is meant those steels to which alloying elements have been added for their effect on physical rather than on electrochemical properties. The effect of cyclic stress-range may be studied by comparing graphs in Figs. 1 to 4 that represent the effect of corrosion at various cyclic stresses, for 20 million cycles, at 1450 r.p.m. Such graphs are represented by the small inverted open triangles and the corresponding light solid lines.

In Fig. 1, the upper row of graphs represents ingot iron and a low-carbon steel; the lower row represents a medium-carbon steel heat-treated in two ways, and an annealed high-carbon steel. In Fig. 2, the two vertical rows on the left represent annealed low-carbon steel, and the vertical row on the right represents quenched-and-tempered steel of slightly higher carbon content. Figs. 3 and 4 represent high-strength alloy steels.

As shown in the figures, stressless corrosion lowers the stage 2 graph considerably below the ordinary endurance graph. The stage 2 graph is lowered about half, sometimes two-thirds, the vertical distance from the endurance graph to the horizontal asymptote of the corrosion-fatigue graph. From the position representing the effect of stressless corrosion, the stage 2 graph is lowered still further as the stage 1 stress is increased. As the stage 1 stress is increased from zero to the corrosion-fatigue limit, the stage 2 graph is lowered from the position representing stressless corrosion to the corrosion-fatigue limit. As shown by all the groups of graphs, however, the lowering of the stage 2 graph with increase in stage 1 stress is not uniform or regular.

To illustrate the change in position of the stage 2 graph with increase in the stage 1 stress, detailed reference will be made to only one group of graphs, the graphs for chromium-molybdenum steel shown in the lower section of Fig. 3. As the stage 1 stress is increased from zero to 6000 lb. per sq. in., the stage 2 graph is lowered only slightly. With further increase in stage 1 stress from 6000 to 8000 lb. per sq. in., the stage 2 graph is lowered much more. With still further increase of stage 1 stress from 8000 to 10,000 lb. per sq. in., the change in position of the stage 2 graph is still greater. With further increase of stage 1 stress, however, from 10,000 to 11,000, 12,000, 13,000 and 18,000 lb. per sq. in., the lowering of the stage 2 graph is hardly perceptible. Increase in stage 1 stress above 18,000 or 19,000 lb. per sq. in. gives variable results. For some specimens after stage 1 stress of 22,000 or 23,000 lb. per sq. in., the stage 2 graph is not much below the graph representing results of stage 1 stresses 10,000 to 18,000 lb. per sq. in. Other specimens stressed at 20,000 lb. per sq. in. fail in stage 1. At stresses above 23,000 lb. per sq. in., all specimens fail in stage 1. The average corrosion-fatigue limit for this material is evidently about 22,000 lb. per sq. in.

All the steels represented in Figs. 1 to 4 show a similar irregular lowering of the stage 2 graph with increase of the stage 1 stress from zero to the corrosion-fatigue limit. This complex relationship between stage 1 stress and stage 2 fatigue limit, however, is better illustrated by a different type of graph, which is derived from the above described stress-cycle graphs and will be called a Type 2 graph. Such graphs are shown in Figs. 7, 8 and 9.

Effect as Illustrated by "Type 2" Graphs:

In the graphs shown in Figs. 7, 8 and 9, abscissas represent stage 1 stress and ordinates represent stage 2 fatigue limit. Each of these Type 2 graphs represents the effect of varying the cyclic stress-range, while keeping the number of cycles and time per cycle constant. The marks used in these figures to designate the number of cycles and kind of water applied in stage 1 are the same as in Figs. 1 to 5. In Figs. 7, 8 and 9, however, each of these marks represents a fatigue limit, whereas in Figs. 1 to 5 each mark represents the result of a single test.

The graphs in Figs. 7 and 8 representing results of prior-corrosion in carbonate water for 20 million cycles will be considered first. The effect of cyclic stress-range on specimens corroded in this way is illustrated by the curved lines drawn through the small inverted open triangles.

Although these graphs represent a variety of carbon and alloy steels with wide range of physical properties, the graphs are surprisingly similar in form. Each graph starts from the axis of ordinates in a nearly horizontal direction. but gradually curves downward until it is apparently approaching a vertical asymptote at an abscissa of 7000 to 9000 lb. per sq. in. Instead of continuing this way, however, the curvature is abruptly reversed at an abscissa of 7000 to 9000 lb. per sq. in.; the graph then becomes nearly horizontal and continues so until it approaches an abscissa corresponding to the corrosion-fatigue limit. At this point the curvature is again reversed and the graph descends almost vertically to the axis of abscissas.

At the corrosion-fatigue limit, as previously stated, the stage 1 stress and the stage 2 fatigue-limit coincide. For this reason, the corrosion-fatigue limit is represented on each graph by a point, designated *C.F.*, which is so placed that its abscissa and ordinate are the same. From this point each graph descends almost vertically to the axis of abscissas. The abscissas at which the graph reaches the axis of abscissas evidently represents the cyclic stress at which a specimen would fail just at the end of stage 1. As this stress is only slightly above the corrosion-fatigue limit, the vertical descent of the graph from the point *C.F.* to the axis of abscissas is in accordance with expectation. Near the point *C.F.* many of the graphs in Figs. 7 and 8 change direction abruptly from nearly horizontal to nearly vertical. In some of the graphs, however, the abrupt change in direction is considerably above the point *C.F.* Possible reasons for this will be given later.

For better comparison of the form and position of the above-described Type 2 graphs, they have been assembled in Fig. 9. In the middle section of the figure are shown the graphs for nickel steel and for all but two of the carbon steels. In the right-hand section of the figure are shown the graphs for the other two carbon steels and the graphs for alloy steels containing chromium.

The ordinates of these graphs at the origin differ greatly and the abscissas at which the graphs reach the axis of abscissas differ considerably. The abscissa at which each graph first approaches a vertical asymptote, however, varies only slightly. In general, this abscissa is possibly a little greater for the steels containing chromium than for the carbon or nickel steels.

Notching Limit and Notch-Depth Limitation:

The gradual and accelerated downward curvature of the Type 2 graph from the origin until it is nearly vertical would naturally be attributed to a gradual and accelerated increase in the depth and sharpness of corrosion pits with increase in the cyclic stress-range. The abrupt reversal of curvature and the nearly horizontal extension of each graph to an abscissa representing much higher stress would naturally be explained on the surmise that with increase of cyclic stress-range above a certain value the depth of corrosion pits increases only slightly. Although microscopic examination of specimens has not kept pace with the prior-corrosion fatigue tests, it has advanced far enough to show that these surmises are essentially correct.

With increase in stage 1 stress from zero there is a gradual and accelerated increase in depth of pitting, from the shallow pits of stressless corrosion to sharp, deep circumferential notches. For this reason, in the previous paper (6) the stress at which the Type 2 graphs first approach a vertical asymptote was called a "notching limit." With increase of stage 1 stress from the notching limit nearly to the corrosion-fatigue limit the depth and sharpness of notches change very little. In future discussion, as in the previous paper (6), this phenomenon will be called "notch-depth limitation."

Notch-depth limitation is probably due chiefly to decrease in intensity of corrosion at the bottoms of the notches with increase in depth. This decrease in intensity is probably due to increase of length of the column of corrosion products through which the electrolyte must diffuse.

For steels, notch-depth limitation becomes effective before, through stress concentration, the actual stress at the bottoms of the notches reaches the endurance limit. Failure, therefore, does not occur at the notching limit. To cause failure, the nominal stress must be raised above the notching limit until the actual stress at the bottoms of the notches reaches the endurance limit. Under such conditions the corrosion-fatigue limit is greater than the notching limit. Notch-depth limitation, therefore, accounts for the outward curvature of the Type 2 graph for steels from the notching limit to the corrosion-fatigue limit.

If notch-depth limitation were absent, a slight increase of stage 1 stress above the notching limit would cause failure. The notching limit for carbon and high-strength alloy steels would then be the corrosion-fatigue limit. Actually, however, for these steels the corrosion-fatigue limit is much higher than the notching limit. It will be shown later, that for some metals the corrosion-fatigue limit and the notching limit coincide.

Although accumulation of corrosion products in the notches is probably the chief cause of the complex form of the Type 2 graph, the form of the graph, for some steels at least, is influenced by the well-known strengthening effect of repeated stresses slightly below the endurance limit. If the initial nominal stress is slightly below the corrosion-fatigue limit, the actual stress at first is below the endurance limit. As notches form and deepen, however, the actual stress at the bottoms of the notches rises and approaches the endurance limit. As the endurance limit is approached, the repeated stress strengthens the metal at the bottoms of the notches and thus raises the endurance limit. The stage 2 fatigue-limit of such a specimen, therefore, is somewhat greater than if there had been no local increase of the endurance limit. Such local elevation of the endurance limit may have the effect of elevating, or at least delaying the descent of, the Type 2 graph as it approaches an abscissa representing the corrosion-fatigue limit. This hypothesis may explain the indication in some graphs, such as the graphs for materials IX-W-9 and IH-16 in Fig. 7, that these graphs actually turn upward just before their final descent to the axis of abscissas. It may also explain the fact that in other graphs, such as the graphs for materials AL-O-10 and BC-W-10 in Fig. 8, the portion to the left of the point *C.F.* is elevated considerably above this point.

Effect of Cyclic Stress-Range on Corrosion of Stainless Iron in Carbonate Water

Results of prior-corrosion fatigue tests of stainless iron after four different heat-treatments are shown in Fig. 5. For comparison, there are also shown results of fatigue and corrosion-fatigue tests. The graphs representing fatigue and corrosion-fatigue are similar to the graphs for this material presented in previous papers(1,2,4). Though the endurance limits, as shown by the four graphs in Fig. 5, vary from 45,000 to 66,000 lb. per sq. in. in proportion to the strength of the material, the corrosion-fatigue limits differ comparatively little. The hardest material, IB-W-10, however, has the lowest corrosion-fatigue limit, and the corrosion-fatigue graph for this material shows the greatest scatter of individual results. This is possibly due in part to internal stress.

In the investigation of the effect of cyclic stress-range on corrosion of stainless iron, the conditions of prior corrosion were the same as in the above-described investigation of the effect of cyclic stress-range on corrosion of carbon and high-strength alloy steels. Results of the investigation are represented in Fig. 5 by the small, inverted open triangles. As indicated by the positions of these triangles in Fig. 5, and by the adjacent letters, the behavior of stainless iron under corrosion, with or without cyclic stress, is very different from the above described behavior of carbon and ordinary alloy steels. Prior corrosion in carbonate water lowers the stage 2 fatigue-limit little (if at all) below the endurance limit, unless the stage 1 stress approaches the corrosion-fatigue limit. This is evidently true for stainless iron in all the degrees of hardness represented in Fig. 5, though the effect of cyclic stress on the hardest material is somewhat obscured by the wide scatter of results.

The relationship between stage 1 stress and stage 2 fatigue-limit is illustrated by the Type 2 graphs shown at the left of Fig. 9. As shown in this figure, each graph representing results of corrosion in carbonate water starts at a point little if at all below the point representing the endurance limit. As each graph extends to the right it descends very little until it approaches an abscissa representing the corrosion-fatigue limit. The graph then curves rapidly downward and descends almost vertically to the axis of abscissas.

The Type 2 graph for stainless iron, therefore, differs greatly from the graph for carbon and ordinary alloy steels. The graph for stainless iron starts at a point much nearer to the point representing the endurance limit than does the graph for carbon or high-strength alloy steels. The graph for stainless iron shows no evidence of reversal of curvature. Its first approach to a vertical asymptote is at an abscissa representing the corrosion-fatigue limit.

For stainless iron in carbonate water, therefore, pitting is evidently negligible at cyclic stresses below a fairly definite limit, which may be called either "pitting limit" or "notching limit." With this limit the corrosion-fatigue limit coincides. As shown in Fig. 9, the notching limit for stainless iron in carbonate water is nearly four times as great as for carbon and ordinary alloy steels.

Effect of Cyclic Stress-Range on Corrosion of Steels and Stainless Iron in Soft Water

Steels:

From the carbonate water used in the above-described experiments calcium carbonate is deposited on steel. As pointed out by Mr. John R. Baylis in discussing a previous paper by the author(3), this deposit of calcium carbonate tends to protect steel against ordinary corrosion. It seemed desirable, therefore, to use a softer water in additional prior-corrosion fatigue tests, and to compare the results with the above-described results obtained with carbonate water. For these experiments, use was made of artesian well water from the U. S. Naval Academy. This water has been softened by chemical treatment. Its composition is given in Table IV.

Results of prior-corrosion fatigue tests with carbon and high-strength alloy steels in soft water are shown in Figs. 2, 4, 5, 7 and 8. As illustrated by the graphs in Figs. 7 and 8, the results obtained with soft water are practically the same as the above-described results obtained with these steels in carbonate water. As illustrated by the graphs for materials BC-W-10 and IW-W-10 in Fig. 8, the soft-water graph and the carbonate-water graph practically coincide throughout the entire length. The corrosion-fatigue limit obtained with soft water is little if any lower than that obtained with carbonate water. For carbon and high-strength alloy steels, therefore, calcium carbonate deposited from water evidently has little influence on notch-depth limitation and gives very little protection against corrosion-fatigue.

Stainless Iron:

For stainless iron also, as shown in Fig. 5, results obtained with the one unbroken specimen of material IB-W-11 indicate that the damaging effect of soft water is little if at all greater than the damaging effect of carbonate water.

EFFECT OF CYCLIC STRESS-RANGE ON CORROSION OF STEELS AND STAINLESS IRON IN SALT WATER

Steels:

Results of prior-corrosion fatigue and corrosion-fatigue tests of carbon and high-strength alloy steels in salt water are represented in Figs. 2, 4, 7, and 8 by the solid triangles and (in Figs. 2 and 4) corresponding light solid lines. Results of corrosion-fatigue tests in salt water are shown in Fig. 1. The salt water used in these experiments is Severn River water of the composition given in Table IV.

As shown in Figs. 7 and 8, the graphs for salt water practically coincide with the graphs for carbonate water until the graphs reach an abscissa representing the notching limit. At this point the graph for salt water reverses its curvature, as does the graph for carbonate water, and turns outward. The outward curvature for the salt-water graph, however, is much less abrupt than for the fresh-water graph. Consequently the salt-water graph descends below the fresh-water graph. For some steels, such as IW-W-10 in Fig. 8, the salt-water graph curves outward until it is nearly horizontal, then again reverses its curvature and descends nearly vertically to the axis of abscissas. For other steels, such as JA-W-11.75 in Fig. 7, the salt-water graph at the notching limit curves outward very little, then reverses its curvature and descends nearly vertically to the axis of abscissas.

For all three kinds of water, therefore, the effect of cyclic stress between zero and the notching limit is about the same. Notch-depth limitation, however, is much less effective in salt water than in the other two kinds of water. The difference between the notching limit and the corrosion-fatigue limit, therefore, is less for salt-water corrosion than for fresh-water corrosion. It seems possible, moreover, that if the time or number of cycles in stage 1 were increased, or if the salt concentration were increased, the corrosion-fatigue limit and the notching limit for these steels in salt water might coincide.

Stainless Iron:

Results of prior-corrosion fatigue tests on stainless iron in salt water are shown in Figs. 5 and 9. As illustrated by the graphs in Fig. 9, the salt-water graph at its origin is only slightly below the corresponding fresh-water graph. The downward curvature is greater, however, and the approach to a vertical asymptote is at a smaller abscissa for the salt-water graph than for the fresh-water graph. For the salt-water graph, as for the fresh-water graph, there is no evidence of reversal of curvature. The corrosion-fatigue limit apparently coincides with the notching limit.

For stainless iron, the notching limit is considerably lower in salt water than in fresh water. In this respect stainless iron apparently differs from carbon and ordinary alloy steels. It is possible, however, that for carbon and ordinary alloy steels, as well as for stainless iron, the salt-water graph and the fresh-water graph gradually separate as they extend to the right. Such separation might be inconspicuous at the low notching limit of carbon or ordinary alloy steel, although it becomes conspicuous at the relatively high notching limit of stainless iron.

STRESS-CONCENTRATION RATIO

For comparison of the effects of cyclic stress, graphs representing various ferrous and non-ferrous metals have been assembled in Fig. 10. In these graphs, which will be called Type 3, abscissas represent cyclic stress-range and ordinates represent stress-concentration ratio. Type 3 graphs, as well as Type 2 graphs, illustrate the effect of varying cyclic stress-range, with constant time and number of cycles. Type 3 graphs are especially suitable for use in comparing the behavior of metals under stressless corrosion.

The "stress-concentration ratio" is an index of the stress concentration due to corrosion pits. It is obtained by dividing the endurance limit of the metal by the fatigue limit of the corroded specimen. The stress-concentration ratio of uncorroded specimens, therefore, is 1, and the ordinate scale starts at 1.

The ratio thus obtained is a true index of stress-concentration only if the endurance limit of the metal at the bottoms of the pits remains unchanged. If the endurance limit of these regions has been raised by the cyclic stress, the actual stress-concentration ratio would be somewhat greater than the calculated value. In the calculated value there is also an error due to neglect of the decrease in effective diameter of the specimen due to the pitting. In specimens of the size used in the investigation, however, this error is evidently small.

EFFECT OF CYCLIC STRESS-RANGE ON CORROSION OF NON-FERROUS METALS

The previous paper(6) presented results of investigation of effect of cyclic stress-range on corrosion of some alloys of copper, of nickel, and of aluminum. It was shown that the effect varies greatly with the character of the alloy.

Nickel-copper alloys under corrosion in fresh or salt water, behave as do corrosion-resistant steels in fresh water. Pitting is negligible at stresses below a fairly definite limit. For these alloys the corrosion-fatigue limit corresponds to the pitting limit. This behavior is illustrated by the graph for monel metal, material IU-8, in Fig. 10. As the graph for this material extends to the right it remains at ordinate 1 (the axis of abscissas) until it approaches an abscissa representing the corrosion-fatigue limit. The graph then curves rapidly upward until it becomes vertical. The graphs for stainless iron are similar.

For aluminum alloys, pitting is severe at zero stress. For some aluminum alloys, pitting at zero stress is so severe that notch-depth limitation is encountered. For such alloys the damaging effect of stressless corrosion is nearly as great as the effect of simultaneous corrosion and cyclic stress at the corrosion-fatigue limit.

The behavior of three alloys of aluminum is illustrated by the upper three graphs in the right-hand section of Fig. 10. The uppermost graph represents duralumin, the graph next below represents commercial aluminum, and the lower graph of the three represents an alloy containing a small proportion of manganese.

The graphs for duralumin and commercial aluminum, by their horizontal extension from the origin, give evidence of notch-depth limitation even at zero stress. The graph for the less corrodible aluminum-manganese alloy, however, starts lower than the other two graphs and curves upward from the origin. For this alloy, the notching limit is apparently not far from the corrosion-fatigue limit.

INFLUENCE OF ELECTROCHEMICAL AND PHYSICAL PROPERTIES ON THE BEHAVIOR OF METALS UNDER CORROSION AND CYCLIC STRESS

In the left-hand section of Fig. 10 are shown graphs representing carbon steels and a 3½-per-cent nickel steel. In the middle section are shown graphs representing steels containing chromium. A comparison of the graphs in these two sections indicates that the ordinate of each graph at its origin depends somewhat on the hardness of the material. The graph for the quenched-and-tempered chromium-vanadium steel AZ-O-7.5 for example, is above the graph for the annealed material AZ-15.75. The corrosion resistance of an alloy, however, evidently has greater influence than the hardness in determining the ordinate of the graph at the origin. The graphs for steels that contain even a small percentage of chromium, for example, are as low at the origin as the graphs for much softer carbon steels. The graph for high-chromium steel starts much lower than do any of the other graphs for steels.

The influence of corrosion resistance is made still more apparent by comparison of the graphs for aluminum alloys with the graphs for steels. The graphs for two aluminum alloys start much higher than the graphs for much harder steels. The graph for the aluminum-manganese alloy starts as high as the graphs for much harder steels. Evidently, therefore, the stress-concentration ratio after stressless corrosion depends chiefly on the electrolytic solution pressure of the alloy, but is considerably influenced by the hardness of the material.

The effect of physical properties on the corrosion-fatigue limit depends on the amount of notch-depth limitation. If, through notch-depth limitation, the corrosion-fatigue limit is greater than the notching limit, the corrosion-fatigue limit is more influenced by physical properties than if the corrosion-fatigue limit coincides with the notching limit. The notching limit is little affected by any physical properties, except possibly the modulus of elasticity. As will be shown in Part III, however, the notching limit for any metal is not a constant. It is influenced somewhat by time and by number of cycles.

PART III.—EFFECT OF TIME AND NUMBER OF CYCLES ON CORROSION

General Description of "Type 4" Graphs:

Consideration must now be given to the effect of time and number of cycles on corrosion. In considering the effect of these two factors, however, it is necessary to keep in view the effect of the third factor, cyclic stress-range, and to study the interrelationship of all three factors. The time factor, moreover, must be studied both as time per cycle and as total time.

To study the effect of time and number of cycles on corrosion, results of prior-corrosion fatigue tests, presented in Figs. 1 to 6, have been selected, and graphs derived from these have been assembled in Figs. 11 and 12. Abscissas in Figs. 11 and 12 represent number of cycles and total time, and ordinates represent stress-concentration ratio. Each graph in these figures represents the effect of varying number of cycles (and hence of varying total time) with constant cyclic stress-range and constant time per cycle. These Type 4 graphs are assembled in groups so that comparison of the graphs in any group will reveal the influence of either cyclic stress-range or time per cycle.

The Form and Position of "Type 4" Graph as Influenced by Cyclic Stress-Range:

Each graph in Figs. 11 and 12, with the exception of two graphs in Fig. 12, may be divided into two parts. In the first part the slope is relatively steep; in the second part it is much less steep. As ordinates are an index of depth and sharpness of pits or notches, the tangent of the angle of slope represents the change in depth and sharpness per cycle. The word "penetration" will be used to designate increase in depth and sharpness.

The first part of each graph, therefore, represents relatively great penetration per cycle; the second part represents decreased penetration per cycle due to notch-depth limitation. In some of the graphs in Figs. 11 and 12, there is evidence of actual increase of slope from the origin to an abscissa of about 8 million cycles. This would seem to indicate that the penetration per cycle increases for about 8 million cycles, then rapidly decreases.

In Fig. 11, each graph illustrates the effect of varying number of cycles, with constant cyclic stress range, at 1450 r.p.m. The group of graphs at the right of Fig. 11 represent results obtained with chromium-molybdenum steel, the same steel that is represented in Figs. 5 and 8. This group of graphs consists roughly of three divisions. In the middle division, the slope of each graph beyond 8 or 10 million cycles is much greater than in the upper and lower divisions. At an abscissa of 20 million cycles, this middle division is widely separated from the upper division; at an abscissa of 50 million cycles the middle division is most widely separated from the lower division. As will be shown later, this change in the relative position of the middle division with increase in number of cycles from 20 to 50 million corresponds to a decrease in the notching limit from about 9000 to 7000 lb. per sq. in.

Comparison of the slopes of the steeper parts of the graphs at the right of Fig. 11 shows that the slope increases with increase in stress. This indicates that the penetration per cycle increases with increase of stress. The influence of increase of stress on the slope of the graph, and hence on the penetration per cycle, is greatest at the notching limit.

Similar relationships for low-carbon steels are shown in the two groups of graphs at the left of Fig. 11. These will not be discussed in detail.

The Form and Position of "Type 4" Graph as Influenced by Time per Cycle:

Each graph in Fig. 12 represents the effect of varying number of cycles, and varying total time, with constant cyclic stress-range, and constant time per cycle. In each of the two groups in the figure, different graphs represent different times per cycle (cycle frequencies). Hence, by comparison of the graphs in each group, it is possible to study the effect of time per cycle on the form and position of the Type 4 graph.

The group of graphs at the left of Fig. 12 shows results obtained with the same chromium-molybdenum steel that is represented in Fig. 11. These graphs represent results obtained with stress of 13,000 lb. per sq. in., at cycle frequencies of 100, 500 and 1450 r.p.m. The lowest graph of this group has also been shown in Fig. 11 and has been described. Each of these graphs consists of a steep portion and a nearly horizontal portion connected by a curve. This curve is less abrupt for low than for high-cycle frequency.

A comparison of these three graphs shows that the slope of the first part of the graph increases with increase in time per cycle. The penetration per cycle, therefore, increases with increase in time per cycle. As indicated by a comparison of the tangents of the angle of slope, however, the increase in penetration per cycle is less than in proportion to the increase in time per cycle. Moreover, as the time per cycle is increased, the effect of this increase on the penetration per cycle evidently grows continually less.

The time rate of penetration, however, as shown by a comparison of these three graphs, increases with decrease in time per cycle (increase in cycle frequency). The increase in time rate, however, is less than in proportion to the increase in cycle frequency. Moreover, as the cycle frequency is increased the effect of this increase on the time rate of penetration evidently grows continually less. The above-described relationship between time per cycle, penetration per cycle, and time rate of penetration applies only to the first, relatively steep, part of the Type 4 graph.

The influence of time per cycle on the upper part of the Type 4 graph must now be considered. As illustrated by the graph at the left of Fig. 12, the greater the time per cycle the greater is the elevation of the nearly horizontal part of the graph, and hence the greater is the indicated notch depth for constant number of cycles. This conclusion seems to be well established. The slope of this part of the upper two graphs, however, is based on relatively few experiments, each of which required much time. Further experiments are necessary, therefore, to establish definitely the slope of the upper parts of these graphs. If these slopes in Fig. 11 are correct, however, they lead to rather surprising conclusions, which are indicated in the figure by the broken lines representing constant time. These lines indicate that the notches, if given sufficient time at low cycle frequency may actually be deeper than if given the same time at high cycle frequency.

Such a relationship would not be inconsistent with the fact that, in the relatively rapid penetration before depth limitation is reached, the time rate of penetration increases with decrease in time per cycle (increase in cycle frequency). It should be noted that the greater the time per cycle the greater is the time required to reach depth limitation. At cycle frequencies of 1450, 500 and 100 r.p.m., the notches reach nearly maximum depth at about 4, 14, and 60 days, respectively. At high cycle frequency, though the initial time rate of penetration is rapid, this rate soon slackens abruptly. At low-cycle frequency the initial rate is maintained longer and slackens less abruptly; hence the notches reach greater depth. This hypothesis will be tested by additional experiments.

The group of graphs at the right of Fig. 12 represents results obtained with a chromium-vanadium steel that had been heat-treated to produce high strength. Comparison of the graphs in this group confirms the above-expressed conclusion as to the relationship between time per cycle, penetration per cycle, and time rate of penetration before depth limitation is reached.

The upper parts of the graphs in this group are not so nearly horizontal as those that have been previously described. Notch-depth limitation is less prominent for this hard material than for the softer steels previously discussed. Nevertheless, it should be noted that the sensitivity of the ordinate scale is great, and that the sensitivity increases with increase in the ordinate. This accounts in part for the slope in the upper parts of the graphs at the right of Fig. 12.

Effect of Time and Number of Cycles on the Notching Limit:

In discussing Fig. 11, it was pointed out that the notching limit is affected by number of cycles (and total time). The effect of time and number of cycles on the notching limit, however, is better illustrated by Type 2 graphs as represented in the left-hand section of Fig. 8. These graphs each represent the effect of varying cyclic stress-range, with constant time per cycle and number of cycles. With increase in the number of cycles, the graph is moved downward and to the left. The notching limit, therefore, decreases with increase in the number of cycles applied in stage 1. As indicated both by this figure and by Fig. 11, however, with increase in the number of cycles the effect of such increase on the notching limit probably grows continually less.

The notching limit probably decreases also with increase in time per cycle. This subject has not been investigated.

The notching limit, therefore, must be viewed as a variable. It decreases with increase in time and number of cycles. Nevertheless, as shown in Part II, it is little affected by physical properties, but is affected by the corrosion resistance of the metal.

PART IV.—EFFECT OF DIAMETER OR THICKNESS OF SPECIMEN ON THE CORROSION-FATIGUE LIMIT

Probable Effect of Decrease of Diameter or Thickness of Specimen Below ½-in.:

For specimens with diameter or thickness less than about ½ in. the corrosion-fatigue limit has not as yet been investigated. Results of investigation of the effect of cyclic stress-range on corrosion as presented in Part II, however, lead to some conclusions as to the probable effect on the corrosion-fatigue limit of decreasing the specimen to small diameter or thickness. This subject was discussed briefly in the previous paper(6).

It was shown in Part II of the present paper that, for carbon and ordinary alloy steels, the notching limit is considerably below the corrosion-fatigue limit. This conclusion, however, applies only to specimens whose diameter or thickness is large compared with the limiting depth of notches. With specimens whose diameter or thickness is small compared with the limiting depth of notches, the effect of decreasing section with increase in depth of notch would become prominent. Under these conditions, the rise of actual stress with increase in notch depth would be much greater than for the relatively large specimens used in the investigation described in Part II. It seems probable, therefore, that with decrease in diameter or thickness of carbon or ordinary alloy steel specimens, the actual stress at the notching limit would approach the endurance limit; hence the corrosion-fatigue limit would approach the notching limit. If the size of specimen is sufficiently decreased, the corrosion-fatigue limit would be no higher than the notching limit, which for carbon and ordinary alloy steels is 8000 or 9000 lb. per sq. in. For metals that are deeply pitted at zero stress, moreover, the corrosion-fatigue limit would be reduced to zero, if the diameter or thickness of specimen is reduced until the pits of stressless corrosion reach the axis of the specimen.

The corrosion-fatigue limit of metals that are deeply pitted by stressless corrosion, therefore, and especially of metals for which notch-depth limitation is prominent, is much decreased by thinning the specimen. For low-potential alloys which are not appreciably pitted by corrosion at zero stress, or at any stress below the corrosion-fatigue limit, the effect of thinning section is probably slight.

Description of Investigation of Effect of Increase of Diameter of Specimen:

To investigate the effect of considerable increase in diameter of specimen, corrosion-fatigue tests have been made with specimens having diameters of 1.7 to 2.3 in. The larger diameters were used in tests at the lower stresses. For these experiments, cylindrical specimens held in a lathe were revolved in contact with a stream of water. The diameter of the part gripped in the lathe averaged about 0.4 in. greater than the diameter of the part that was subjected to corrosion-fatigue. Between these two parts was a fillet with $2\frac{1}{2}$ in. radius. The distance from the bottom of the fillet to the plane of application of load was 24 in. The specimens were revolved at 275 to 400 r.p.m., the greater velocity being used at the lower stresses. To avoid the effect of stress concentration at the fillet, the water stream was applied 2 in. out from the fillet. The water was allowed to drop on the specimen so as to cause corrosion in a relatively narrow ring around the specimen. The method of polishing for these large specimens was the same, and the surface finish was as smooth, as for the small specimens. Under corrosion-fatigue these specimens always failed at the corrosion ring, never at the fillet.

For comparison with the tests made with the large specimens, corrosion-fatigue tests were made on small specimens of the ordinary tapered form described in previous papers. These small specimens were machined from the large bars.

Results of Experiments with Heat-Treated Chromium-Nickel Steel:

The chromium-nickel steel bars used in these experiments were all heat-treated by the Wyman-Gordon Co. to meet the same specification for crankshafts. The chemical composition and physical properties are given in Tables I and III. Results of experiments with this steel are illustrated in Fig. 13 by the five graphs on the left. Curves 1, 2 and 3 represent results obtained with the ordinary tapered specimen that has been described in previous papers. The diameter of this specimen at the region of maximum stress is a little less than $\frac{1}{2}$ in. Curve 1 represents results of ordinary endurance tests in air. Curve 2 represents results of corrosion-fatigue tests in carbonate water at 1450 r.p.m. Curve 3 represents results obtained in carbonate water at 250 to 400 r.p.m. Curves 4 and 5 represent results obtained with large specimens in carbonate and salt water, respectively. The large specimens were tested at 250 to 400 r.p.m., the lower frequency being used at the higher stresses.

A comparison of Curves 2 and 4 shows that the corrosion-fatigue graph for large specimens is below the corrosion-fatigue graph obtained with small specimens at 1450 r.p.m. As it appeared possible that this difference in position is due in part to difference in time per cycle, additional small specimens were tested by corrosion-fatigue at 250 to 400 r.p.m., the cycle frequencies that were used in testing the large specimens. The results, represented by Curve 3, indicate that, even when tested at the same cycle frequency, the graph for large specimens is slightly below the graph for small specimens. Decrease in the size of the stream of water causes no further lowering of Curve 3.

The results of experiments with this steel, therefore, seem to indicate that the corrosion-fatigue limit for the large specimens is slightly below the corrosion-fatigue limit for small specimens. A conservative conclusion, however, is that for this hard steel increase of diameter of specimen from $\frac{1}{2}$ to 2.3 inches does not raise the corrosion-fatigue limit.

The corrosion-fatigue limit of the large specimen in carbonate water is about one-fourth the endurance limit. In salt water the corrosion-fatigue limit has not been determined accurately but as indicated by Curve 5 is probably not more than 10,000 lb. per sq. in.

Results of Experiments with Low-Carbon Steel:

To determine whether or not results btoained with soft steel would be similar to the above described results obtained with the chromium-nickel steel, additional experiments were made with steel having a carbon percentage of only 0.09. This was stock material, the origin of which is not known. The composition, as given in Table I, would indicate that it is bessemer steel.

Results obtained with this low-carbon steel are shown in the three graphs at the right of Fig. 13. Curves 1 and 2 represent results obtained with the ordinary tapered specimens. Curve 1 represents results of ordinary endurance tests in air. Curve 2 represents results of corrosion-fatigue tests in carbonate water at 1450 r.p.m. Curve 4 represents results of corrosion-fatigue tests at 250 to 400 r.p.m.

As shown in this figure, the corrosion-fatigue graph for large specimens is lower than the corrosion-fatigue graph for small specimens. Even when the large and small specimens are tested at the same cycle frequency, the corrosion-fatigue limit is apparently lower for the large specimens than for the small specimens.

The results obtained with this soft steel, therefore, agree with the results obtained with the harder steel. The corrosion-fatigue limit for the larger specimen apparently is slightly lower than for the small specimen. It seems, possible, however, that if the corrosion per cycle could be made exactly the same for both sizes of specimen, the corrosion-fatigue limits would be practically the same. It appears certain that for the large specimens the corrosion-fatigue limit is no greater than for the small specimens.

Conclusions:

These results seem to indicate also that, with still further increase in diameter of specimen, the corrosion-fatigue limit would be no greater. They lead to the conclusion that the corrosion-fatigue limit is practically unaffected by variation in diameter or thickness of specimen, provided the diameter is large compared with the depth limit of notches.

These results also indicate, in view of the results presented in Part II, that stress-concentration due to corrosion notches depends on the depth and sharpness of the notch and not on the diameter of the specimen, provided the diameter of the specimen is large compared with the depth of the notch. They lead to the general conclusion that for carbon and high-strength alloy steels the relation between cyclic stress-range and notch depth (as indicated by a Type 2 graph), the notching limit, and the corrosion-fatigue limit, are independent of the size or thickness of specimen, provided the diameter is large in comparison with the notch-depth limit.

PART V.—TORSIONAL CORROSION-FATIGUE OF STEEL

Object of Investigation:

Information is much needed about torsional corrosion-fatigue of metals, especially about the torsional corrosion-fatigue limit and its relation to the corrosion-fatigue limit obtained by rotating-cantilever test. For this reason, experiments have been made with chromium-vanadium steel heat-treated to produce two different degrees of hardness. By rotating-cantilever tests, the ordinary endurance and corrosion-fatigue graphs for this material had previously been determined and presented in previous papers(**4,6**).

Description of Investigation:

Specimens ½ in. in diameter and 2 in. in length between fillets were subjected to alternating-torsion in machines of the spring type described in a previous paper(**10**). Tests in this machine were made at 350 cycles per minute.

The chromium-vanadium steel was the same that was used in the rotating-cantilever tests described in the previous paper(**6**). The composition, heat-treatment, and physical properties are given in Tables I, II, and III. The alternating-torsion specimens were made from the grip ends of the previously tested rotating-cantilever specimens. The method of finishing these specimens was the same as for the rotating-cantilever specimens.

For corrosion-fatigue tests a small stream of water was allowed to drop on the middle of the specimen. Corrosion by this stream did not extend to the fillets.

Fatigue and Corrosion-Fatigue Graphs:

Results of fatigue and corrosion-fatigue tests by rotating cantilever and by alternating torsion are shown in Fig. 14. The group of graphs on the right represents quenched-and-tempered material, and the group of graphs on the left represents fully-annealed material. Curves 1 and 2 are taken from previous papers(**4,6**). They represent results of rotating-cantilever tests in air and in carbonate water, respectively. Curves 3 and 4 represent results of alternating-torsion tests in air and in carbonate water, respectively.

The torsional corrosion-fatigue tests have been carried far enough to indicate the approximate corrosion-fatigue limit for each material. The results indicate that the torsional corrosion-fatigue limit for either hard or soft material is about 8000 lb. per sq. in. This result was unexpected in view of the fact that the rotating-cantilever corrosion-fatigue limits for materials AZ-15.75 and AZ-O-10 are 22,000 and 16,000 lb. per sq. in., respectively.

It is of interest also to compare for each material the ratios between the two endurance limits and the ratios between the two corrosion-fatigue limits. The ratios between the two endurance limits for materials AZ-15.75 and AZ-O-10 are, respectively, 0.64 and 0.67. The ratios between the two corrosion-fatigue limits for AZ-15.75 and AZ-O-10 are, respectively, 0.37 and 0.50. The ratio between the torsional corrosion-fatigue limit and the rotating cantilever corrosion-fatigue limit for material AZ-15.75, therefore, is unexpectedly low. Possible reasons for this will be discussed after consideration of the results of microscopic examination of the surfaces of specimens.

Results of Microscopic Examination of Specimens After Torsional Corrosion-Fatigue:

Photomicrographs of corrosion-fatigue cracks or notches in rotating-cantilever specimens were shown in previous papers(1,2). These cracks or notches are transverse to the line of stress. Reference has also been made in the previous paper(6) and in this paper to the transverse direction of the notches produced in rotating-cantilever specimens by corrosion under cyclic stress below the corrosion-fatigue limit. It seemed desirable, therefore, to investigate the direction of notches in specimens that had been subjected to torsional corrosion-fatigue.

Under alternating torsion the directions of principal tensile and compressive stress make an angle of 45 deg. with the axis of the specimen. The directions of principal shear stress, however, are longitudinal and transverse. Fracture of hard specimens under alternating torsion in air is usually in a spiral direction, hence perpendicular to the direction of maximum tension and compression. Fracture of softer metals under alternating torsion in air is usually by transverse shear. It is of interest, therefore, to know whether failure under torsional corrosion-fatigue is by tension or by shear.

Examination of the surfaces of specimens that had failed under torsional corrosion-fatigue shows, after removal of the products of corrosion, numerous small x marks. These marks are visible in great number by an ordinary low-power lens. The appearance of these marks at higher magnification is illustrated in Fig. 15. The upper photomicrograph shows the appearance of the marks after only partial removal of the corrosion products. The other photomicrographs show the appearance after complete removal of the corrosion products. These marks are evidently due to cracks intersecting at right angles. The cracks make an angle of 45 deg. with the axis of the specimen and are perpendicular, therefore, to the direction of maximum tension. As the cracks have the same appearance in hard and soft material, they are evidently characteristic of torsional corrosion-fatigue.

Possible Effect of Intersecting Cracks on Notch-Depth Limitation:

The fact that the torsional corrosion-fatigue limit is about the same for quenched-and-tempered and for fully-annealed chromium-vanadium steel may possibly indicate that notch-depth limitation (described in Part II) is less effective under these conditions than under corrosion-fatigue by rotating-cantilever. Under torsional corrosion-fatigue, therefore, the corrosion-fatigue limit may be nearer the notching limit than under rotating-cantilever corrosion-fatigue. The idea suggests itself that notch-depth limitation may be less effective with intersecting than with non-intersecting notches. Additional experiments, however, are needed to confirm or disprove this hypothesis.

ACKNOWLEDGMENTS

Acknowledgment is due to the following personnel of the Naval Engineering Experiment Station: to Capt. H. C. Dinger, U. S. N., Officer in Charge, and Lieut. Comdr. T. A. Solberg, U. S. N., Test Officer, for encouragement received in this investigation; and to Messrs. George F. Wohlgemuth, Associate Metallurgist; Lawrence Thompson, Associate Metallurgist; W. C. Stewart, Assistant Metallurgist; W. E. Harvey, Junior Metallurgist; A. P. Vandermast, Laboratorian; and John K. Amoss, Leadingman-machinist, for assistance in the experiments.

Acknowledgment is also due to the following manufacturers: the American Rolling Mill Co., the Firth Stirling Steel Co., the Wyman-Gordon Co., the International Nickel Co., and the Aluminum Co. of America for generous cooperation by furnishing material.

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-GENERAL LEGEND-

Fatigue.

⊙--*In air, 1450 r.p.m., broken.*

⌖--*In air, 1450 r.p.m., unbroken.*

Corrosion-Fatigue.

△---*In carbonate water, 1450 r.p.m., broken.*

-△---*Ditto; unbroken.*

◭---*In soft water, 1450 r.p.m., broken.*

-◭---*Ditto; unbroken.*

▲---*In salt water, 1450 r.p.m., broken.*

-▲---*Ditto; unbroken.*

Prior-Corrosion Fatigue.

Corrosion stage in water, 1450 r.p.m., at stress and number of cycles indicated by mark and accompanying letter, then fatigue stage in air at 1450 r.p.m., and at stress and number of cycles indicated by position on the graph.

▽--*Prior corrosion in carbonate water, 20 million cycles, broken in fatigue stage.*

-▽--*Ditto; unbroken in fatigue stage.*

□--*Prior corrosion in carbonate water, 8 million cycles; broken in fatigue stage.*

-□--*Ditto; unbroken in fatigue stage.*

◇--*Prior corrosion in carbonate water, 4 million cycles; broken in fatigue stage.*

-◇--*Ditto; unbroken in fatigue stage.*

×---*Prior corrosion in carbonate water, 50 million cycles; broken in fatigue stage.*

⧨---*Prior corrosion in soft water, 20 million cycles; broken in fatigue stage.*

-⧨---*Ditto; unbroken in fatigue stage.*

▼---*Prior corrosion in salt water, 20 million cycles; broken in fatigue stage.*

-▼---*Ditto; unbroken in fatigue stage.*

Stresses in Corrosion Stage

Letter on Graph.	Cyclic Stress (Half Range) lb. per sq. in.
A	*Zero.*
B	*4,000*
C	*5,000*
D	*6,000*
E	*7,000*
F	*8,000*
G	*9,000*
H	*10,000*
I	*11,000*
J	*12,000*
K	*13,000*
L	*15,000*
M	*17,000*
N	*18,000*
O	*19,000*
P	*20,000*
Q	*21,000*
R	*22,000*
S	*23,000*
T	*24,000*
U	*25,000*
V	*26,000*
W	*27,000*
X	*28,000*
Y	*29,000*
Z	*30,000*
a	*32,000*

General Legend for Figures.

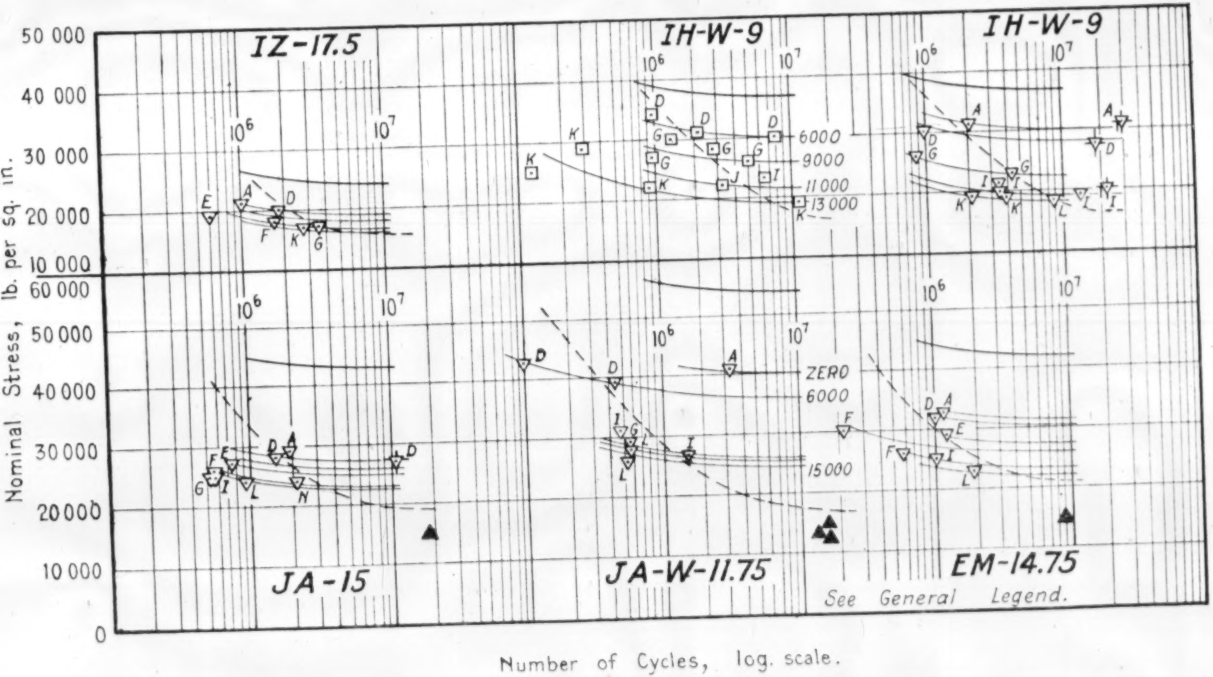

FIG. 1.—Effect of Cyclic Stress-Range on Corrosion of Carbon Steels.

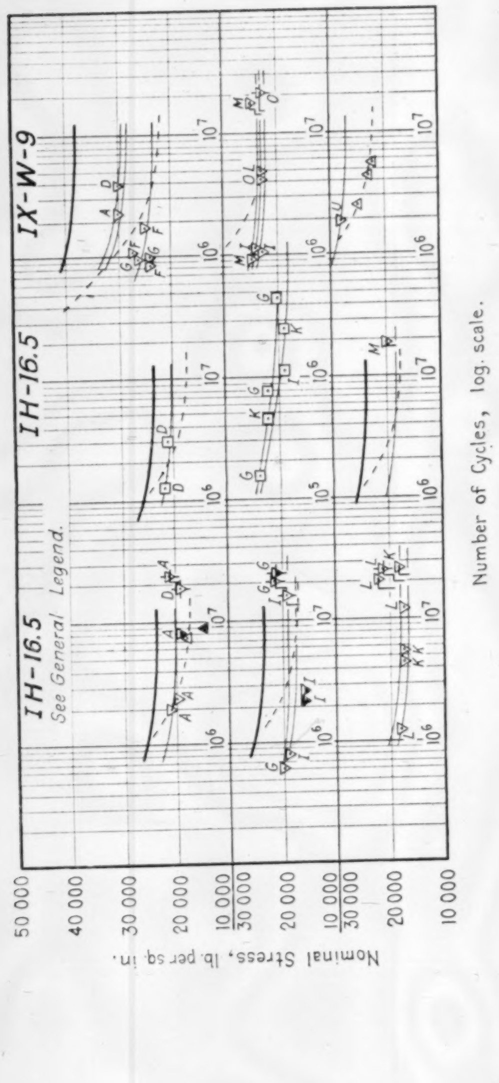

FIG. 2.—Effect of Cyclic Stress-Range on Corrosion of Carbon Steels.

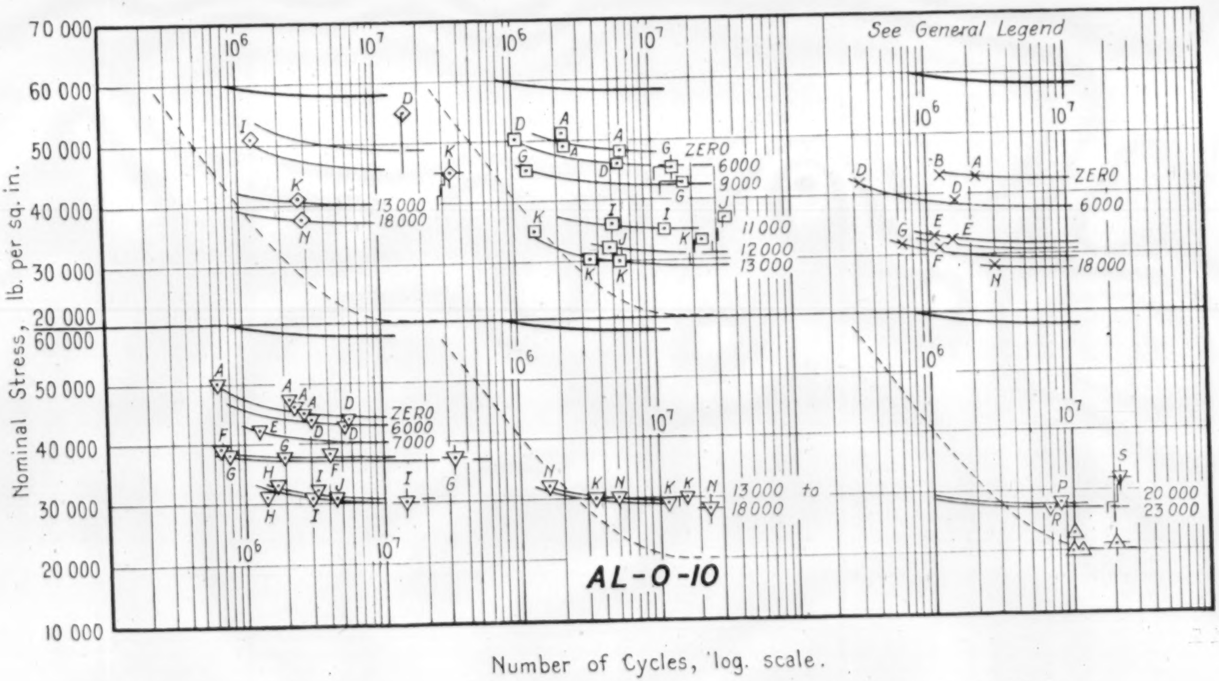

FIG. 3.—Effect of Cyclic Stress-Range on Corrosion of Chrome-Molybdenum Steel.

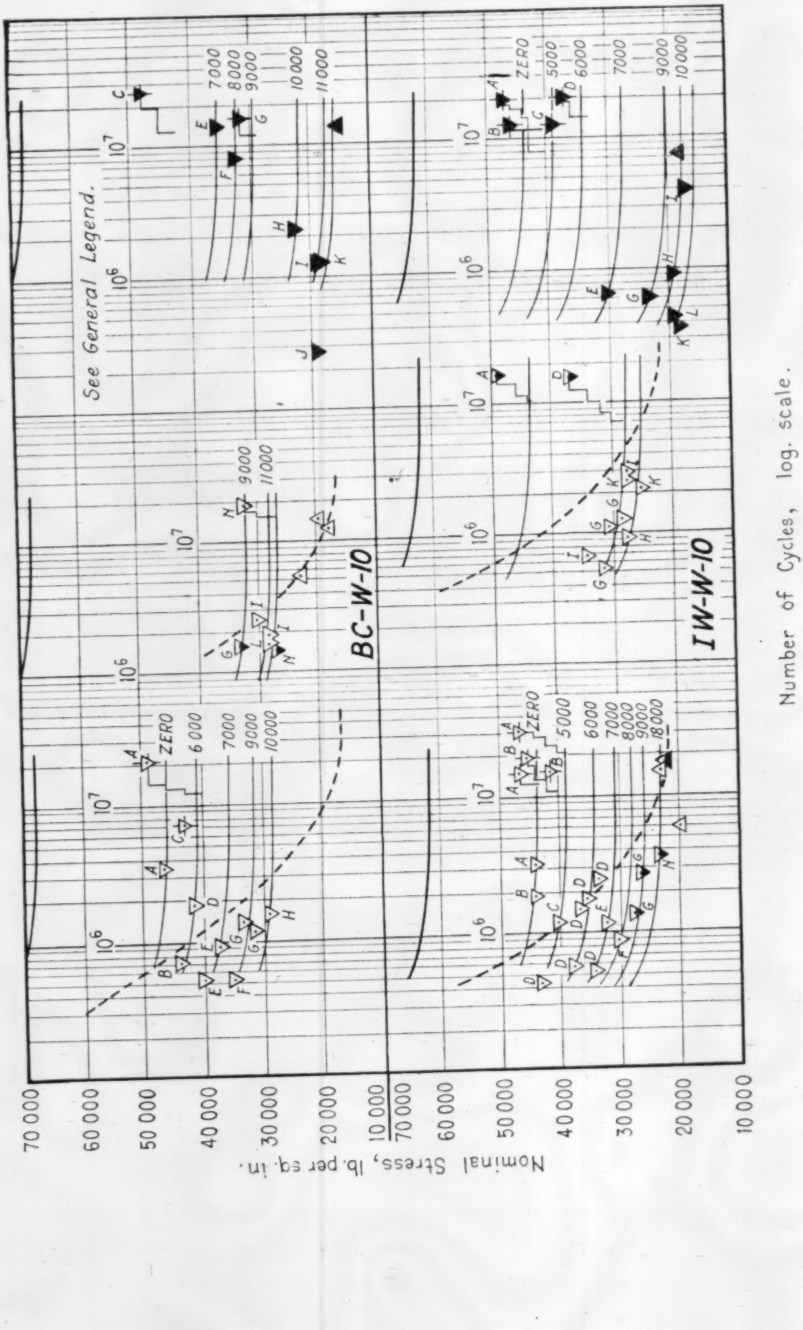

FIG. 4.—Effect of Cyclic Stress-Range on Corrosion of Nickel and Chrome-Nickel-Steels.

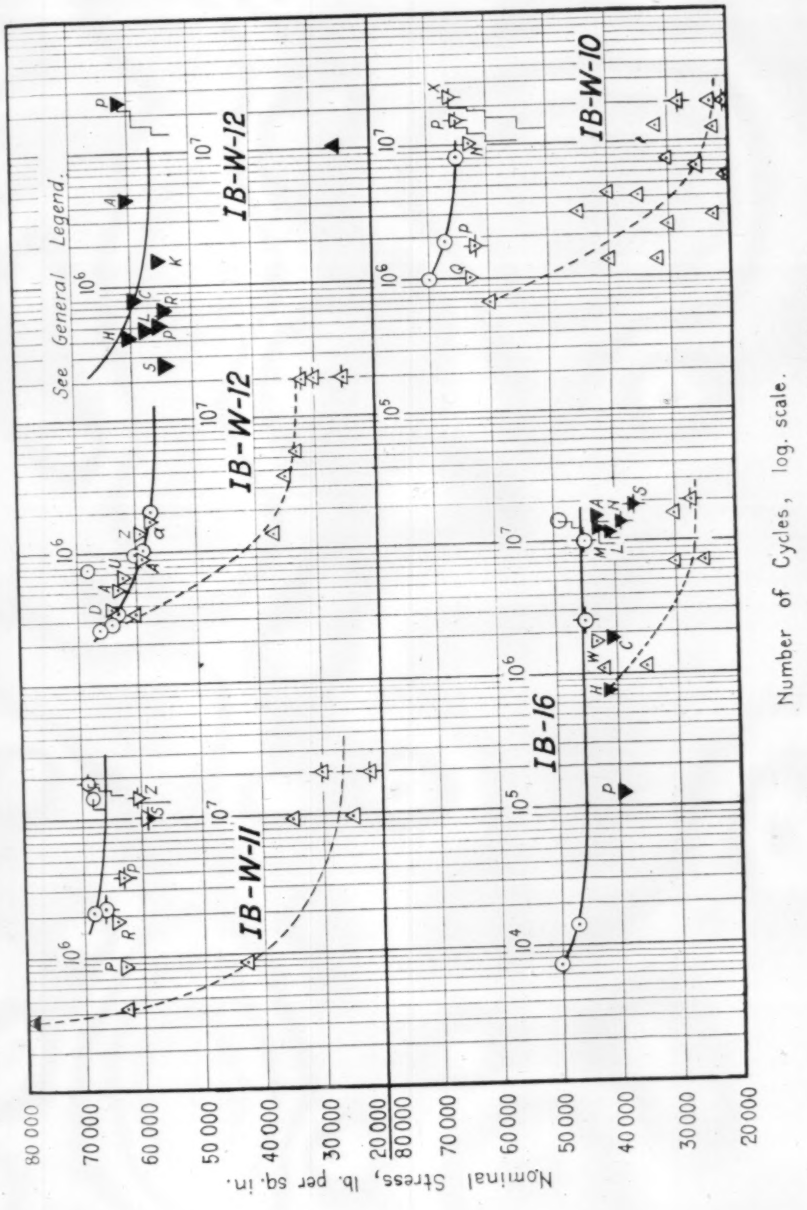

FIG. 5.—Effect of Cyclic Stress-Range on Corrosion of Stainless Iron.

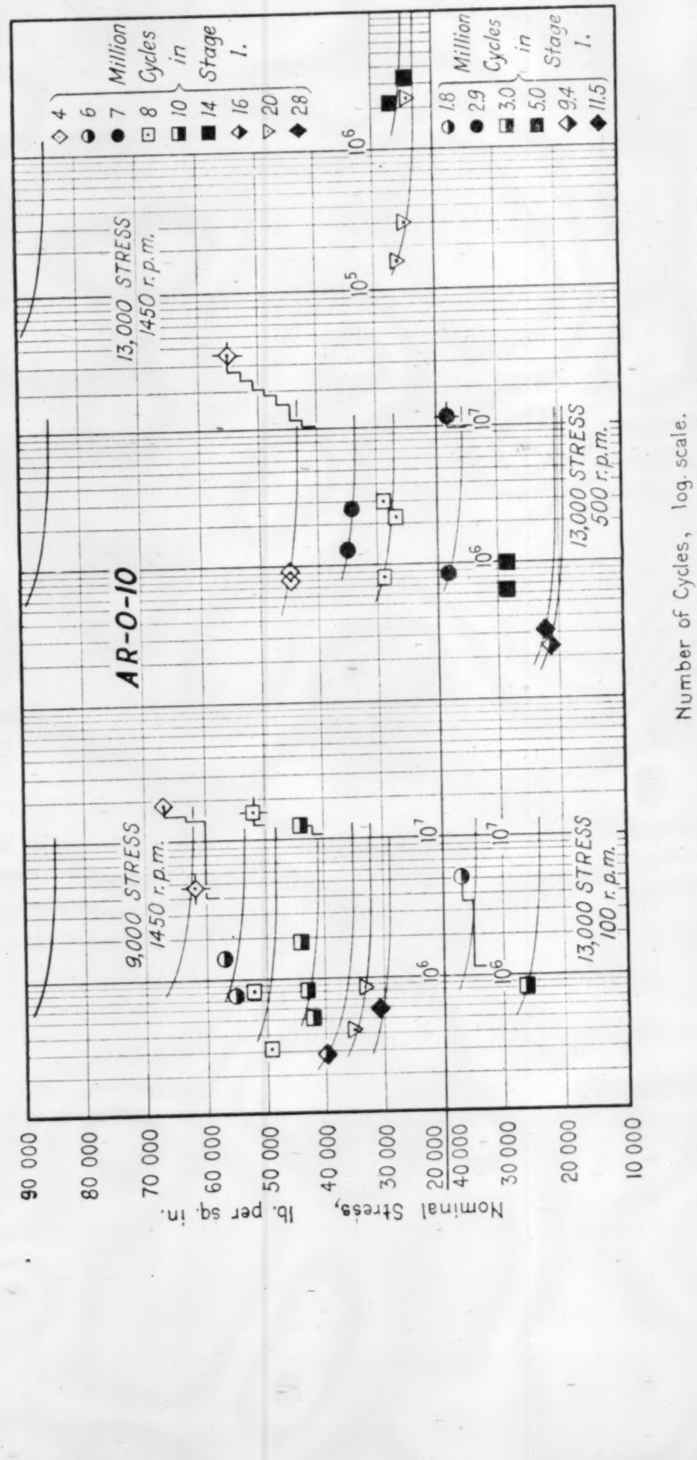

Fig. 6.—Effect of Time and Number of Cycles on Corrosion of Chrome-Vanadium Steel.

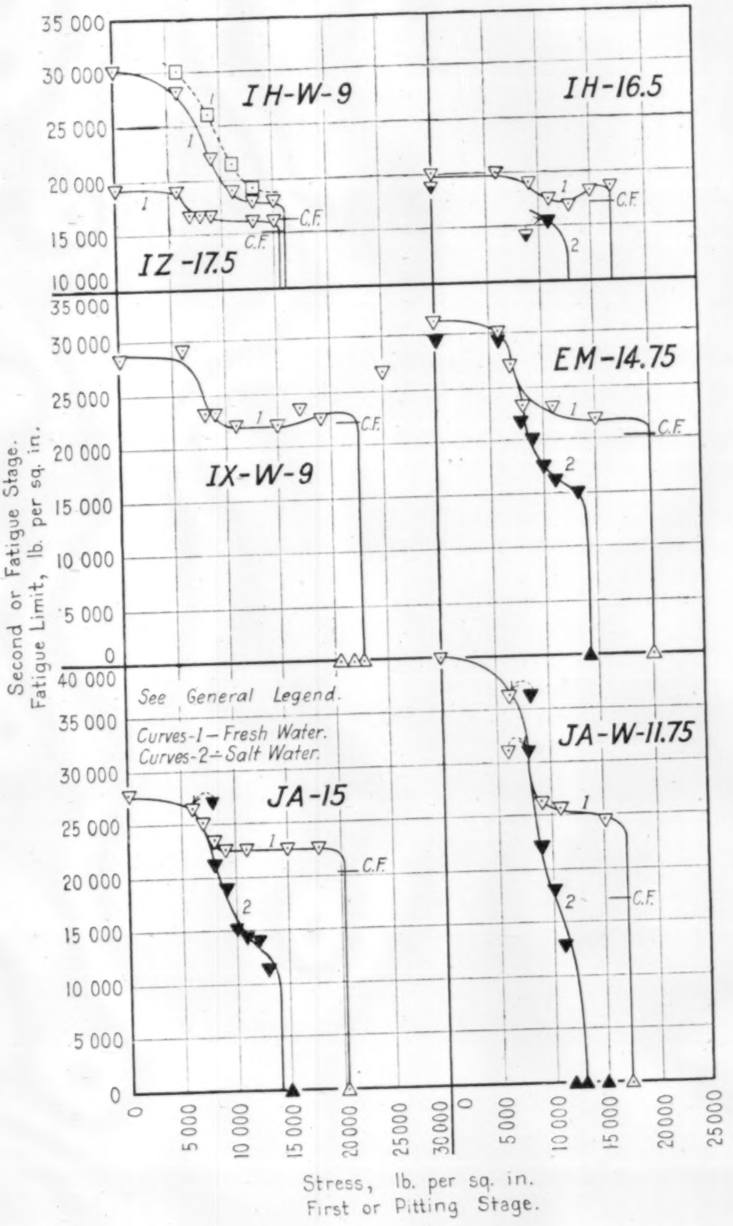

FIG. 7.—Effect of Cyclic Stress-Range on Corrosion of Carbon Steels, Type 2 Graphs.

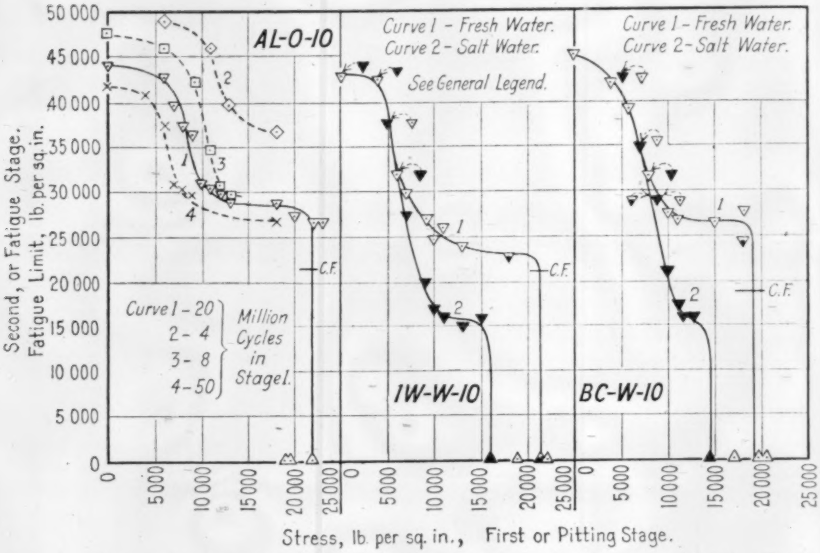

FIG. 8.—Effect of Cyclic Stress-Range on Corrosion of High-Strength Alloy Steels Type 2 Graphs.

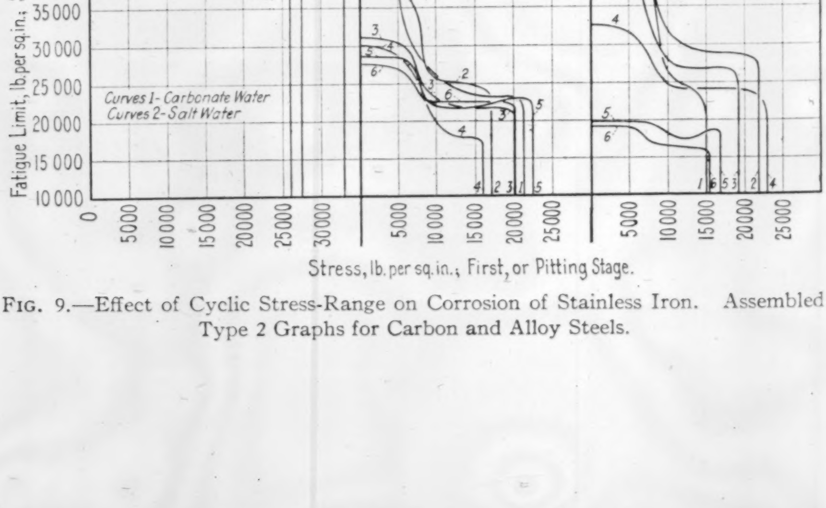

FIG. 9.—Effect of Cyclic Stress-Range on Corrosion of Stainless Iron. Assembled Type 2 Graphs for Carbon and Alloy Steels.

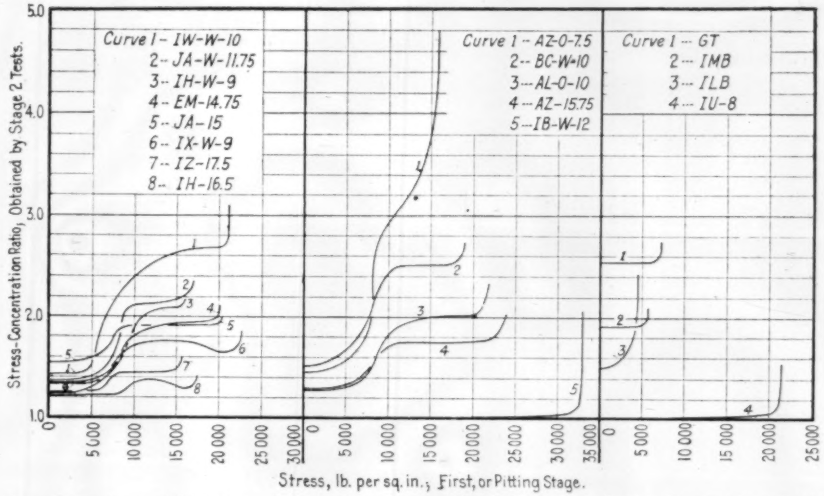

FIG. 10.—Assembled Type 3 Graphs, Illustrating Effect of Cyclic Stress-Range on Corrosion of Steels and Non-Ferrous Metals.

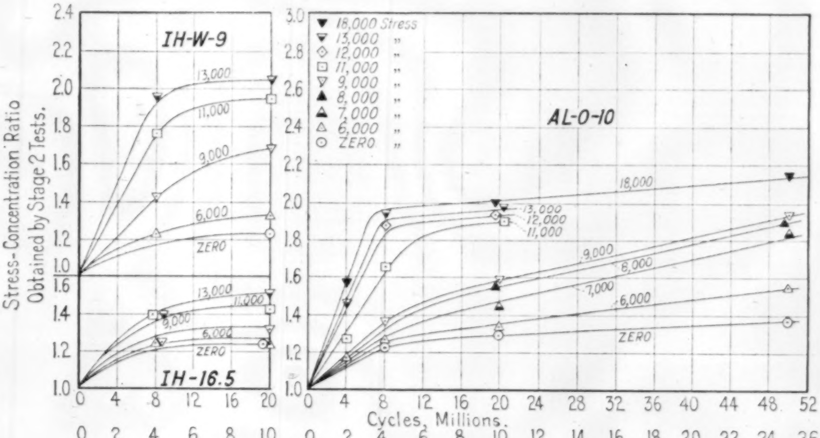

FIG. 11.—Effect of Number of Cycles, Time, and Cyclic Stress-Range on Corrosion of Steel.

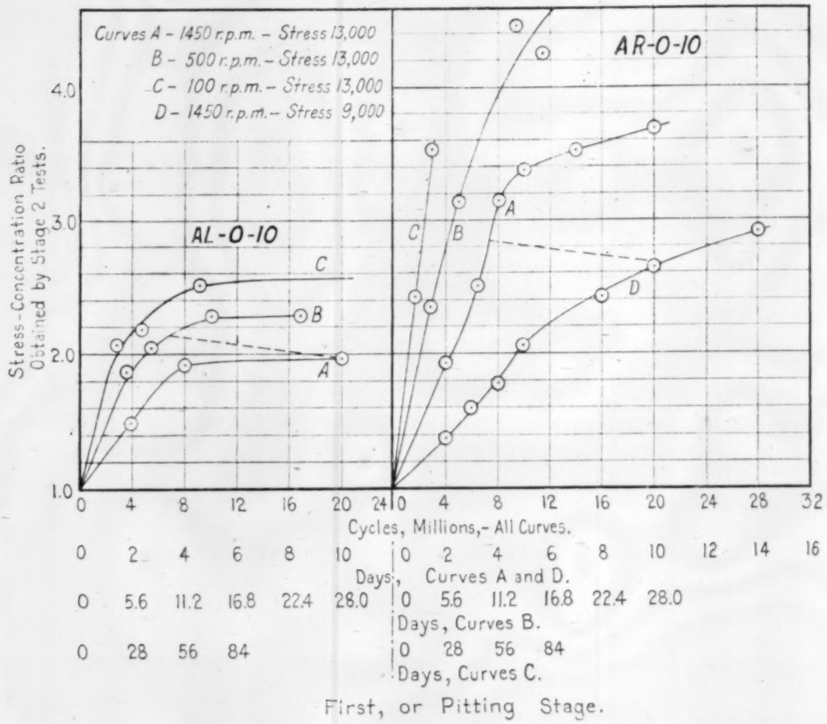

FIG. 12.—Effect of Time and Number of Cycles on Corrosion of Steel.

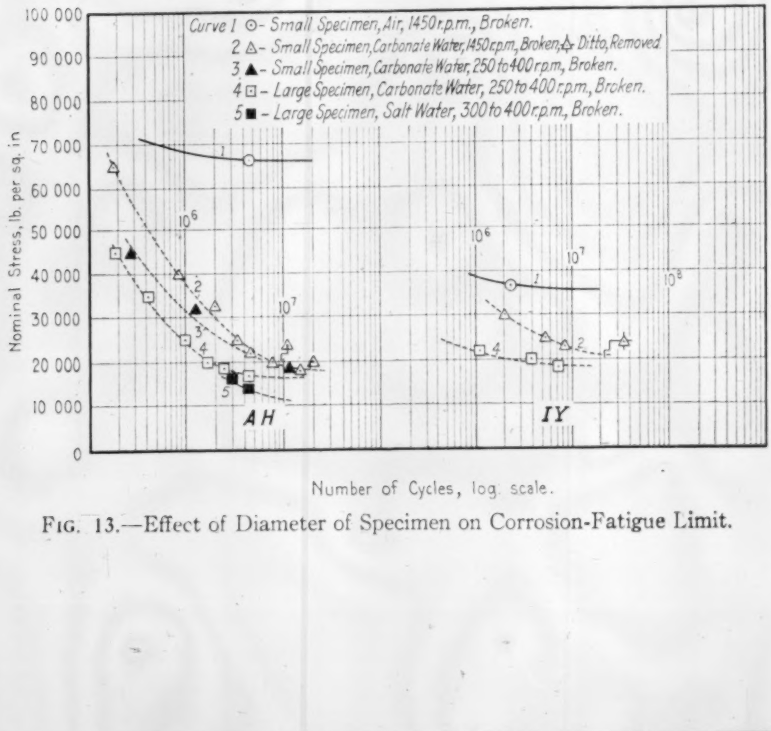

FIG. 13.—Effect of Diameter of Specimen on Corrosion-Fatigue Limit.

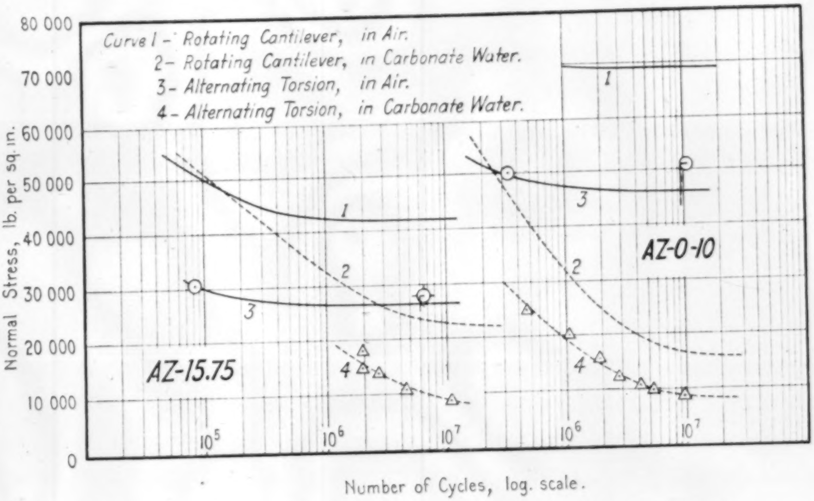

FIG. 14.—Torsional Corrosion-Fatigue of Chrome-Vanadium Steel.

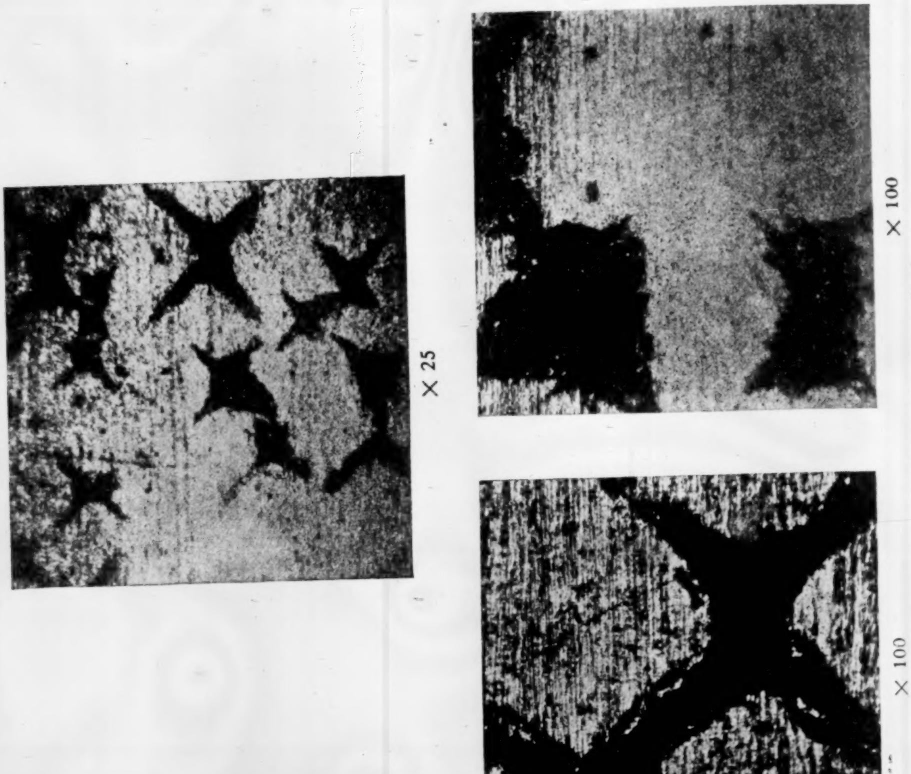

FIG. 15.—Notches on Surface of Specimens, Produced by Torsional Corrosion-Fatigue.

[For Discussion on Corrosion-Fatigue of Metals, see page 168.–Ed.]

FIG. 15.—Notches on Surface of Specimens, Produced by Torsional Corrosion-Fatigue.

INFLUENCE OF CORROSION ACCELERATORS AND INHIBITORS ON FATIGUE OF FERROUS METALS

F. N. SPELLER,[1] I. B. McCORKLE[1] AND P. F. MUMMA[1]

SYNOPSIS

This paper compares ordinary corrosion and corrosion-fatigue with respect to the factors involved, many of which are common to both cases. With this comparison in mind, experiments are described showing the effect of inhibitors on corrosion-fatigue with and without other external influences tending to accelerate corrosion.

For the water and steel used as a basis of this work, it is shown that a small amount of sodium dichromate will prevent the lowering of the air-fatigue limit by corrosion, but where a strong tendency to pitting is introduced (in addition to cyclic stress) a much larger amount of inhibitor fails to prevent a serious reduction in the endurance limit. However, the use of inhibitors under such conditions tends to increase the useful life of the metal under fatigue by slowing up the action of localized corrosion.

Further experimental work is planned on the influence of inhibitors and protective coatings on corrosion-fatigue under various corrosive media.

INTRODUCTION

A study of ferrous metals has indicated definitely that there is a limiting stress, at or below which the material may be cyclically stressed indefinitely without apparent injury,—the so-called "fatigue" or endurance limit. Experiments by D. J. McAdam, Jr.,[2] show that under certain conditions corrosion has a marked influence in lowering the resistance to fatigue of metals under cyclic stresses, especially when corrosion and cyclic stresses occur simultaneously. This has been termed "corrosion-fatigue."

This paper refers to fatigue tests made with sufficient inhibitor in the water to passivify the steel and prevent general corrosion, and to the effect of certain corrosion factors which tend to break down the protective influence of the inhibitor and cause pitting of the metal under stress. Further details of this problem are now being followed up experimentally, so that while we believe the present conclusions are definite, a more complete statement on certain details may be expected later. This work seems to have an important bearing on the preservation of the original factor of safety in structures subject to cyclic stresses.

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[1] Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

[2] D. J. McAdam, Jr., "Stress-Strain-Cycle Relationship and Corrosion-Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 224 (1926); "Corrosion-Fatigue of Metals as Affected by Chemical Composition, Heat Treatment, and Cold Working," *Transactions*, Am. Soc. Steel Treating, March, 1927, Vol. XI, No. 3.

FATIGUE, ORDINARY CORROSION AND CORROSION-FATIGUE

Ordinary fatigue tests in air have established a general relationship between the tensile strength and the endurance limit under fatigue. In these tests corrosion seems to have been a minor factor, but the thermal effect at high stresses has apparently caused a lowering of the fatigue-endurance curve in that range. This may be due in some measure to corrosion in moist air accelerated locally by the higher temperature of the most strained fibers or to direct oxidation at elevated temperatures which occurs when the test specimen is not cooled.

Corrosion of metals in water depends upon a number of conditions which include the following:

1. Factors controlling amount of free oxygen reaching the metal surface, such as temperature, rate of motion, surface coatings, passivifying agents, etc. In the long run corrosion is mainly a question of surface protective films.

2. Dissolved substances which act more or less independently of the dissolved oxygen present, such as carbonic and other acids, alkalies, hydrogen sulfide, etc.

3. Factors influencing localization of corrosion (pitting), such as internal stresses, contact with dissimilar materials, or the influence of electrolytic concentration cells of all kinds.

The composition of ordinary iron or steel within a rather wide range has been shown to have a relatively minor effect on corrosion when the metal is continuously immersed.

Corrosion-fatigue as defined above according to the work of McAdam is markedly distinguished from ordinary fatigue and is controlled by different factors. The results of his experiments (which are confirmed for the material used within the scope of these tests) show: (1) that even slight corrosion with fatigue may cause failure at stresses far below the ordinary endurance limit; (2) corrosion of the unstressed metal prior to fatigue usually does much less damage in lowering the resistance to subsequent fatigue than corrosion *combined* with fatigue; (3) "corrosion-fatigue" depends more on the tendency of the metal to corrosion (especially in the form of pitting) than on its physical properties, heat treatment, or chemical composition, except in so far as the latter have a marked effect on the rate of corrosion; and (4) that cyclic stress greatly accelerates local corrosion when the stress is over a certain amount.

ORDINARY CORROSION AND CORROSION-FATIGUE

It appears then that ordinary corrosion and corrosion-fatigue are primarily influenced by the same factors, but that in the latter case a very active form of local corrosion may be started by cyclic stresses and that ultimate failure in such case depends jointly on the rate of penetration of corrosion, due to stress and other factors, acting simultaneously with ordinary fatigue, due to the maximum fiber stress at the base of the corrosion fissure.

The rate of corrosion penetration under cyclic stress is greatly accelerated by interference with the formation of protective coatings, electrolytic effect of stress or strain and local variations of stress in adjacent sections, variations in temperature, and by electrolytic concentration cells set up in minute fissures in the metal or on the metal surface due to local variations in oxygen concentration. It has been shown before that the initial rate of corrosion (without stress) even in slow corroding solutions is comparatively rapid until a protective coating is formed.[1]

In ordinary corrosion it is generally agreed that the most stressed part is anodic. The lower concentration of dissolved oxygen at the base of a fissure acts in the same way to render the metal more anodic. Cyclic stress above a certain value tends to prevent the formation of natural protective coatings or self-healing films. In corrosion-fatigue we probably have these three tendencies working together and in the same direction to accelerate the rate of penetration. It is no wonder, therefore, that destruction of the metal is so rapid where the stress is sufficiently high to disturb the surface and start pitting. This is indicated by the more rapid drop in the corrosion-fatigue curves compared with the air curve.[2]

A corrosion-fatigue limit is often referred to in the literature. As corrosion-fatigue apparently depends so much on the character of the corrosion media and the nature of the protective films formed, it is hard to conceive of a fixed corrosion-fatigue limit for any metal under all conditions of exposure. Moreover, a water which ordinarily forms a good protective coating and is, therefore, considered as only slightly corrosive, may be much more active in contact with the same metal under cyclic stress. In some media, there may be practically no corrosion-fatigue limit over a long period of time.

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[1] Speller and Texter, "Effect of Alkaline Solutions on Corrosion of Steel Immersed in Water," *Industrial and Engineering Chemistry*, Vol. 16, pp. 393–397 (1924).

[2] One distinctive difference between ordinary fatigue and corrosion-fatigue appears to be that in the former an applied stress may be maintained practically constant while in the latter where the stress is above the point which will start corrosion-fatigue the unit stress increases as corrosion fissures deepen.

As corrosion can no longer be ignored in the use of metals under cyclic stress, it becomes an important question as to how best to prevent such action and thus maintain the factor of safety for which the member was originally designed.

Considerable progress has been made in the study of methods for prevention of corrosion in the absence of stress. The experiments which will be briefly referred to in this and subsequent papers were designed to find out how far preventive measures could be applied to corrosion-fatigue. Generally speaking, ordinary corrosion may be prevented by reducing the activity of the reagent, by the use of suitable protective coatings, or by increasing the corrosion-resistance of the metal. The experiments referred to in this paper have to do with the use of a typical and well-known inhibitor added to the water to prevent lowering of fatigue strength by corrosion; incidentally, these tests extend McAdam's original study of the changes in the stress-cycle diagram when the fatigue specimens are kept cool while under test, as suggested by Gillett.[1]

EXPERIMENTS ON INHIBITION OF CORROSION-FATIGUE

The material used for all tests was taken from a single bloom of basic open-hearth steel of the following analysis:

Carbon	0.35 per cent
Manganese	0.79 per cent
Sulfur	0.028 per cent
Phosphorus	0.014 per cent

The test specimens were machined to the dimensions shown in Fig. 1 from $\frac{3}{4}$-in. diameter rods, which had been hand-forged from a 3-in. diameter hot-rolled bar made from the above-mentioned bloom. The rods were normalized in 22-in. lengths prior to machining by heating to 900° C. for $1\frac{1}{2}$ hours followed by air cooling.

The tests were made on the R. R. Moore rotating-beam fatigue testing machine of recent design, which has been described elsewhere.[2]

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[1] H. W. Gillett, Discussion on Fatigue of Metals, *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 271 (1926).

[2] R. R. Moore, "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

The solutions used in testing flowed by gravity from an open 20-liter glass reservoir through glass tubes which distributed the liquid so as to keep the stressed portion of the test specimen completely surrounded by the solution. The solution was collected after flowing over the test piece and returned to the reservoir by means of a hard-rubber pump. At no time during the test did the solution come in contact with any metal, excepting the test specimen itself. The rate of flow over the test specimen was approximately 110 liters per hour. All solutions were maintained completely saturated with oxygen by bubbling air through them continuously.

The test data are shown in Figs. 2 and 3. In Fig. 2, the various fatigue curves are plotted in the conventional manner using the semi-logarithmic scale method. Curves *C* and *D* are moved one logarithmic cycle to the right with respect to Curves *A* and *B* to avoid confusion in inspecting the individual test points. Curve *A* shows the stress-cycle results for this 0.35-per-cent carbon, 0.79-per-cent manganese steel when tested in air with a thin coating of unmedicated vaseline. Curve *B* is the corrosion-fatigue stress-cycle diagram for this same steel when tested in a solution of 25 p.p.m. each of sodium chloride and sodium sulfate in distilled water. This solution will approximate closely the corrosive properties of Pittsburgh tap water. Curve *C* is the result of testing this steel under the same conditions as were used in obtaining Curve *B*, but with the addition to the solution of 200 p.p.m. sodium dichromate as a corrosion inhibitor. It was in obtaining these curves that it was found necessary to introduce the $\frac{7}{16}$-in. long cylindrical portion of the test specimen, shown in Fig. 1. Rubber washers were required near the shoulders of the specimen to keep the solution out of the bearings of the machine. The concentration of inhibitor was not sufficient to prevent corrosion of the test

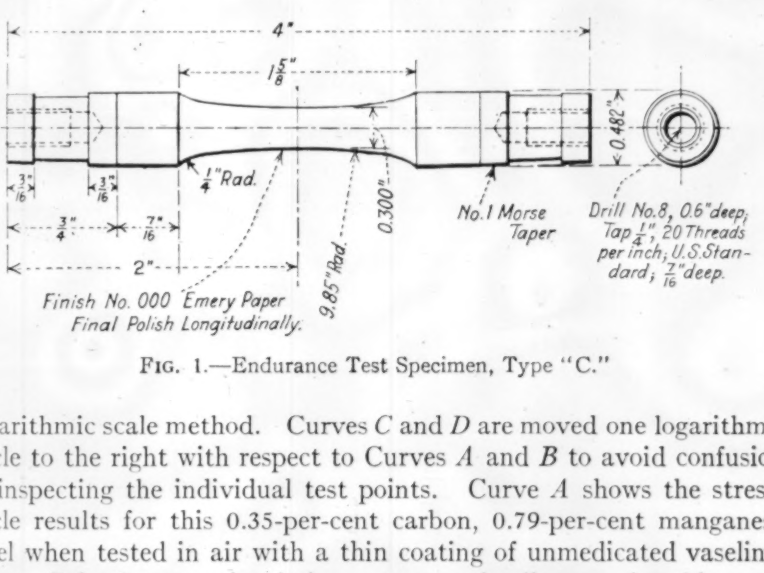
4"
1 5/8"
0.482"
3/16"
3/16"
1/4" Rad.
0.300"
3/4"
7/16"
2"
9.85" Rad.
No. 1 Morse Taper
Drill No. 8, 0.6" deep; Tap 1/4", 20 Threads per inch; U.S. Standard; 7/16" deep.
Finish No. 000 Emery Paper
Final Polish Longitudinally.

FIG. 1.—Endurance Test Specimen, Type "C."

specimen at the washer, with the result that the specimens broke at or near the shoulder of the specimen. By introducing this new cylindrical portion of larger diameter, over which the washer was fitted, the stress concentration was roughly only one-quarter of that at the center, and the failure of the test specimens thereafter took place at the center unless intentionally produced elsewhere by localizing the corrosion at another section of the test specimen.

Curve *D* was obtained by using the same solution as was used for Curve *C*, but in this case the test specimen was painted with a band of clear Duco $\frac{1}{16}$ in. in width at the center. Duco was used because it is inert to steel and, therefore, any failure to inhibit corrosion-fatigue would be due to oxygen concentration cells making the stressed metal at the Duco band more anodic and therefore breaking down the protective film of the inhibitor at that section.

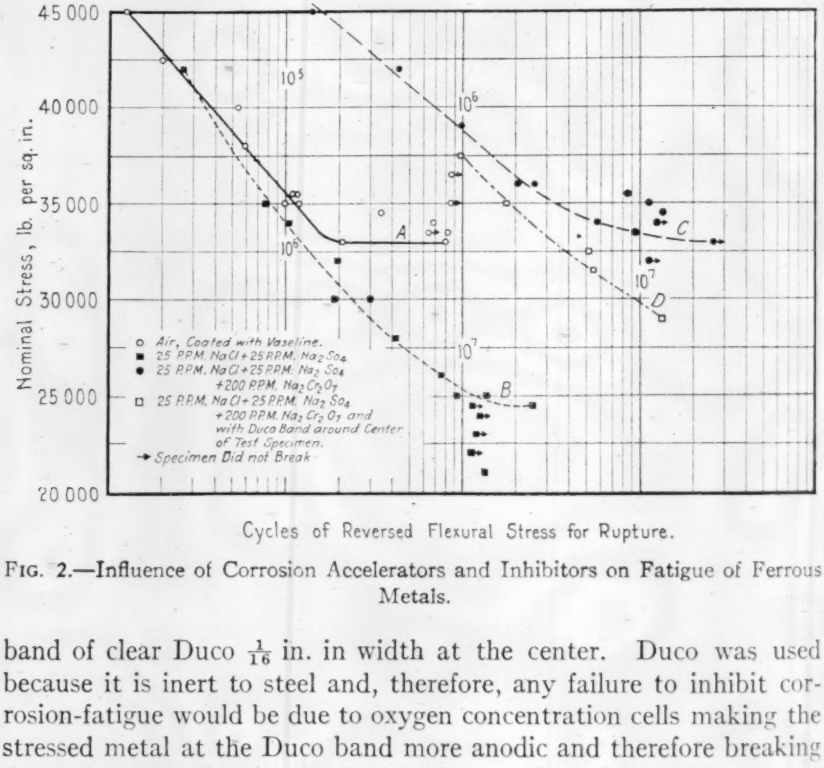

FIG. 2.—Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals.

For facility of inspection of the results, these curves are re-plotted as Curves *A*, *B*, *C* and *D* in Fig. 3, without showing the individual points, and to the same set of axes. A few points of additional tests are also shown. These will be referred to later.

Curves *A* and *B* are in accordance with results previously reported by McAdam on steel not greatly different from the one used in our tests. Curve *C* (inhibitor) is not coincident with Curve *A* (air) but is definitely displaced to the right; that is, the specimens will withstand a greater number of stress cycles when they are kept cool without corrosion. It is also significant that the endurance limit is the same in both cases. Curve *C* shows that 200 p.p.m. sodium dichromate is enough to prevent lowering of the fatigue limit due to general corrosion by the particular water used in this investigation. This is probably closer to the true form of the endurance curve in the absence of corrosion.

Curve *D* shows that this protection can be overcome by factors which tend to cause localization of corrosion. That the tendency towards protection is present is obvious from the position of Curve *D*, lying as it does between Curves *C* and *B*.

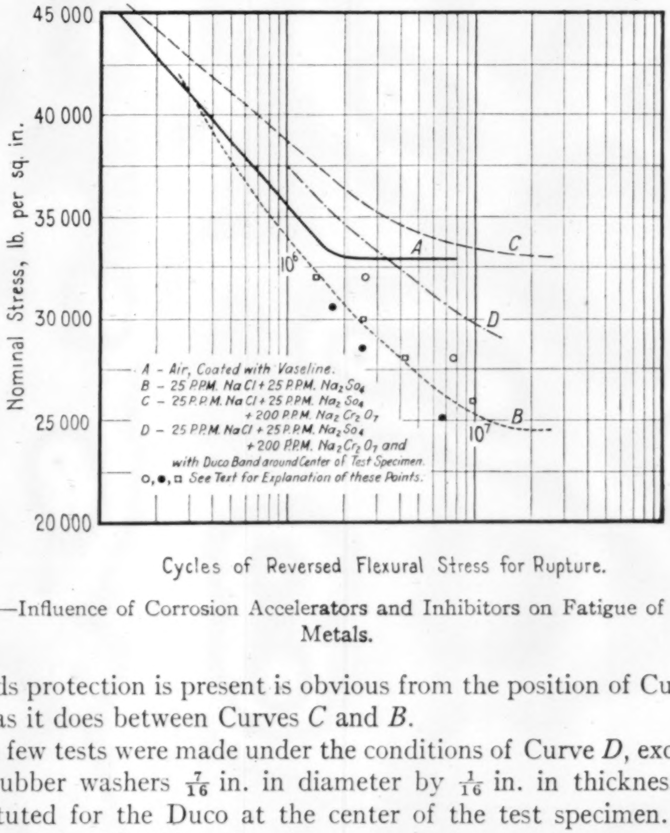

FIG. 3.—Influence of Corrosion Accelerators and Inhibitors on Fatigue of Ferrous Metals.

A few tests were made under the conditions of Curve *D*, excepting that rubber washers $\frac{7}{16}$ in. in diameter by $\frac{1}{16}$ in. in thickness were substituted for the Duco at the center of the test specimen. The points obtained in this series of tests are shown as open squares along Curve *B* in Fig. 3. Here, the conditions which tended to localization of corrosion are much more severe and have completely broken down the protective effect of the inhibitor, as the points practically coincide with Curve *B*. We do not know as yet whether enough inhibitor (within reason) could be added to this water to overcome such a strong tendency to local corrosion. For example, the points shown as open circles to the right of Curve *B* were obtained with a rubber washer in the center, using, however, 1600 p.p.m. inhibitor instead of 200 p.p.m.

Within certain limits, failure of the test specimen can be caused to take place at any desired location. The points represented as solid circles to the left of Curve *B* illustrate this. This series was made using 200 p.p.m. inhibitor and a rubber washer on the test specimen. In each case the calculated stress at the center of the test piece was 32,000 lb. per sq. in., but the rubber washer in the three tests after the first was displaced varying distances from the center. In each case, failure occurred at the washer. The following table gives the result of this series of tests:

	CALCULATED STRESS		
SAMPLE	AT CENTER, LB. PER SQ. IN.	AT WASHER, LB. PER SQ. IN.	DISTANCE OF WASHER FROM CENTER, IN.
No. 1	32 000	30 600	0.21
No. 2	32 000	28 500	0.34
No. 3	32 000	25 100	0.50

CONCLUSIONS

Factors other than fatigue frequently determine the useful life of parts of structures or mechanisms subjected to alternating stresses. In these cases fatigue failures may be avoided by increasing the endurance of the member *for a given stress cycle*, even though the actual stress be above the endurance limit of the material under the prevailing external conditions. It, therefore, appears important to pay especial attention to conditions which will increase the number of cycles before failure occurs, and it is for this reason that we have stressed the significance of changes in slopes and locations of the endurance curves and have considered less prominently changes in the endurance limits under these varying conditions.[1] For example, in the present experiments at a stress of 29,000 lb. per sq. in. with water corrosion, specimens ran about 2,000,000 cycles under the worst conditions and 14,000,000 cycles under the best.

For practical purposes, improvement in physical properties of steel affords increased resistance to fatigue *in the absence of corrosion*, but it is a question how much of this increased value will remain when the stressed part is also subject to considerable corrosion. Therefore, it would seem important to develop as much protection as possible by the use of inhibitors or other means, even if absolute protection cannot be obtained. For example, rotary drill pipe often fails in a way that has not been satisfactorily explained. As both the outside and inside of the pipe are in contact with circulating mud-laden water, it would seem worth while to dissolve an inhibitor in the water where it is more or less corrosive, to increase the number of cycles the pipe will stand, even if enough inhibitor cannot be added to eliminate all the effects of localized corrosion on fatigue. It is evident that protection in practice must be obtained in most cases by the use of suitable protective coatings. Further experimental work is planned on the influence of inhibitors and protective coatings on corrosion-fatigue under various types and intensity of corrosion.

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[1] No attempt is made in this particular investigation to separate the effect of time.

The conclusions at which we have arrived from the work already done are summarized as follows:

1. These experiments confirm previous experimental work by McAdam as to effect of corrosion on fatigue of steel.

2. In these tests sufficient inhibitor (sodium dichromate) to stop corrosion in the presence of fatigue has been shown to prevent reduction of the air fatigue endurance limit when the metal is subject simultaneously to cyclic stress and ordinary corrosion.

3. External factors which tend to localize corrosion have a dominating influence in determining the point of failure under corrosion-fatigue. This effect can be neutralized to a certain extent by the use of inhibitors of sufficient strength.

4. These results indicate the need of developing protective coatings that will not break down in service under cyclic stresses.

DISCUSSION ON CORROSION-FATIGUE OF METALS

Mr. Fletcher.

MR. H. W. FLETCHER[1] (*presented in written form*).—This subject of corrosion-fatigue is most interesting in relation to fatigue failures of rotary drill pipe which is commonly operated in fluids of a corrosive nature. In particular it points to a possible explanation of the opinion widely held among practical drillers that drill pipe run in "salt-water" wells rapidly "crystallizes," that is, becomes brittle.

The localizing effect produced by the rubber washers and by the Duco band mentioned in the paper by Messrs. Speller, McCorkle and Mumma may have some bearing on the fact that fatigue failures in drill pipe are frequently observed in the last thread engaged on the pipe. Tool joints and couplings are commonly assembled on the pipe with a grease lubricant, usually a special grease carrying suspended zinc dust as an anti-galling agent. Resultant potential differences might readily accelerate corrosion with concentration of failures in the region adjacent to the end of the thread engagement, in spite of the fact that the wall section at this point is approximately two and one-half times the thickness of the body of the pipe.

In the practical case the analysis of drill pipe failures is complicated by the existence of complex stresses due to the design of the joint and the nature of loading which can hardly be evaluated. This will inevitably make the demonstration of the corrosion-fatigue effect in drilling wells a most difficult problem.

Mr. McAdam.

MR. D. J. MCADAM, JR.[2]—The paper by Messrs. Speller, McCorkle and Mumma is a very welcome contribution to knowledge of corrosion-fatigue. Except general commendation of the paper, I have few comments. I do not agree with the form of some of the curves in Fig. 2. The form of the curves is of some importance, especially as the authors compare the forms of curves *A* and *C*. Such curves, in my opinion, should be drawn through the center or mean position, rather than through the lower edge of the band of scattered points. If drawn in this way, curve *A* would have a less abrupt approach to the horizontal, and the indicated endurance limit would be slightly higher. The lower portion of curve *C*, also, would be slightly elevated. Curves *A* and *C* in Fig. 3, moreover, would be less widely separated.

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[1] Chief Engineer, Hughes Tool Co., Houston, Tex.

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

Mr. McAdam.

In the last paragraph on page 161, the authors raise the question whether there is a corrosion-fatigue limit under all conditions of corrosion. This depends, however, on the definition of corrosion-fatigue limit. In previous papers I have used the term to mean the ordinate at a point where the corrosion-fatigue curve has become nearly horizontal. Such corrosion-fatigue limits are readily obtained from most corrosion-fatigue graphs. They have been found very useful in comparing metals and corrosion conditions. This does not mean that the corrosion-fatigue curve ever becomes exactly horizontal. For easily corrodible metals under severe corrosion conditions, moreover, the corrosion-fatigue limit may, as the authors say, be rather indefinite.

Mr. Speller.

MR. F. N. SPELLER.[1]—We are glad that Mr. McAdam has explained what he means by "corrosion-fatigue limit." This is no doubt a useful term so long as it is not misunderstood.

(*Presented in written form*).—Mr. McAdam, in his paper, has again given us a comprehensive view of the work done and new developments in his laboratory on corrosion-fatigue factors which is of great interest. Undoubtedly, the effect of corrosion cannot be ignored wherever cyclic stresses occur in structural members. The subject is treated in such a comprehensive manner as to offer few openings for questions. However, the list of important factors involved is not complete with respect to corrosion of metals in general. Perhaps the author intended to limit this list to corrosion-fatigue which seems more proper. In ordinary corrosion and corrosion-fatigue, in addition to the factors mentioned, we have to consider the influence of contact effect of dissimilar metals, concentration cells of all kinds, factors affecting the oxygen concentration at the metal surface and the rate of motion of the corroding solution in contact with the metal surface, the protective influence of films and coatings, and many other minor influences.

Mr. McAdam's experiments on the influence of the three types of water are very interesting. The soft water and the carbonate water, judging from the data presented, show about the same influence on corrosion-fatigue, although in stressless corrosion a soft water would be expected to corrode the metal much more rapidly than a carbonate water of a composition likely to deposit a protective scale. The Severn River water, at Annapolis, is a mixture of sea water and a water carrying considerable calcium bicarbonate, and is the same as that used in the immersion tests being carried out at Annapolis by the Society's Committee A-5 on Corrosion of Iron and Steel, so that it is interesting to compare the results of the static tests which have been going on for several years with tests made with this water

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[1] Director, Department of Metallurgy and Research, National Tube Co., Pittsburgh, Pa.

Mr. Speller. by McAdam under cyclic stress. The committee's ferrous immersion tests on No. 22 gage sheets in this water showed an average life of 1266 days before penetration whereas in the relatively soft and pure water in Washington, D. C. the average life of the same material was considerably less—876 days.[1] In February, 1928, all of the No. 16 gage sheets at Washington had failed, whereas at that time 132 out of 150 specimens at Annapolis had failed, so that the Severn River water without cyclic stress would be classed as a relatively non-corrosive water, apparently due to the combined influence of a number of factors.[2]

The difference in corrosive characteristics of domestic waters varies mainly with their variable scale-forming properties. Apparently this factor may not play a controlling part in corrosion-fatigue due to the disturbing effect of cyclic stresses which prevents the formation of protective coatings. On the other hand, we have shown that the stress does not prevent the formation of protective films in a chromate solution and that in the absence of other disturbing factors a passivifier will operate just as well in the presence as in the absence of stress in retarding corrosion. Protection films so formed are ordinarily invisible but have recently been isolated and examined by U. R. Evans by generating hydrogen gas under the film and thus lifting it off the metal surface. Such films are extremely thin and are so adherent as to suggest that they may be bonded to the atomic structure of the metal. Fatigue tests may afford a new and useful means of studying and testing protective coatings and passivity. The fatigue test combined with corrosion thus establishes a definite distinction between protective coatings of measurable thickness as formed in some waters by the presence of certain constituents in combination with products of corrosion, and the much thinner films formed by passivifying agents.

Mr. McAdam has confirmed his previous conclusions expressed last year[3] "that the effect on corrosion-fatigue limit of the physical properties determined by the usual tension and impact tests is almost negligible in comparison with the effect of corrosion resistance." He cites a case of soft bessemer steel with a corrosion-fatigue limit somewhat superior to a chrome-nickel steel of much higher tensile strength. (Fig. 13, page 156.) The addition of 1 or 2 per cent of nickel or chromium or both, or heat treatment, does not appear to materially

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[1] 1926 Report of Committee A-5 on Corrosion of Iron and Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part I, p. 131 (1926).

[2] 1928 Report of Committee A-5 on Corrosion of Iron and Steel, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 148 (1928).

[3] D. J. McAdam, Jr., "Corrosion-Fatigue of Non-Ferrous Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 102 (1927).

Mr. Speller. affect the corrosion rate in water. Where corrosion and severe alternating stresses occur together for some time, it would appear that little is to be gained by using the more expensive grades of steel or by heat treatment. I should like to ask Mr. McAdam for his view on this question. In the absence of corrosion, or for a comparatively limited number of stress cycles with or without corrosion, proper heat treatment and the use of alloy steels are, of course, often justified by their superior physical properties. It is remarkable how small is the amount of corrosion needed to nullify these properties in expensive steels.

Mr. Lessells. MR. J. M. LESSELLS.[1]—Mr. McAdam uses the term "stress-concentration ratio." I am wondering if this term is not unfortunate, in view of the fact that stress-concentration ratios and stress concentration factors are used to give a measure of the stress on a machine part which, as we know, has no reference at all to the strength of that particular machine part. I can see here some conflict of terms.

A further point in the paper is with reference to the tests made on large fatigue test bars. I am wondering if we have not a scale effect? Most fatigue tests made by Mr. McAdam, and others, have been carried out on test bars $\frac{1}{2}$ in. in diameter. He is now talking about test bars 2 in. in diameter, and it is of great importance to engineers to know that there is no scale effect. In other words, will we get the same values for large specimens as for these comparatively small specimens? Before drawing final conclusions on his corrosion data, it would be very interesting to have some comments from Mr. McAdam as to whether there have been sufficient data obtained from the large fatigue test specimens.

A third point is the effect of deformation. We have here, and also in the paper by Messrs. Speller, McCorkle and Mumma, quite a considerable amount of data on the corrosion of test bars subjected to bending. However, we must not lose sight of the fact that many machine parts are subject to certain fluctuations of stress where they are not subjected to bending stresses. I am wondering, therefore, if Mr. McAdam feels that these large reductions in strength due to corrosion would also apply provided the stresses be not bending but direct tension and compression. The maximum stresses will be the same in both cases, but we know for example that a bend test shows more readily the effects of embrittlement than the tension test.

Mr. McAdam. MR. MCADAM.—In reply to Mr. Speller's reference to my list of factors involved in corrosion, I would call his attention to the words

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

Mr. McAdam. "at least" at the beginning of that list; that was not intended to be a complete list. I agree with Mr. Speller that, under corrosion conditions such as those described in my paper, there is no advantage in using high-grade steel unless the corrosion resistance of the steel has been increased by the alloying or heat treatment.

Mr. Lessells raised some interesting questions. He points out that "stress-concentration ratio" is sometimes used with a somewhat different meaning from that used in the paper. In the paper the term is used to designate the ratio of the endurance limit to the fatigue limit after corrosion. It has been used by others, however, to designate the stress concentration due to change of section, as calculated or as measured optically. The "stress-concentration ratio," as thus obtained, is usually higher than that obtained by fatigue tests. Though the term has been used in these two different meanings, however, its meaning as used in the paper is clear, and thus is justifiable in the absence of a better term.

Mr. Lessells suggests that there may be a "scale effect" or size effect in ordinary fatigue. This idea had been considered by the author in an attempt to account for the fact that the corrosion-fatigue limit for the large specimens is actually a little lower than for the small specimens. To test this idea, we intend to make some ordinary fatigue tests with large specimens of low-carbon steel; large specimens of stronger steel could not be tested in the machine that we have been using. In the meantime, however, the conclusions given in the paper, that there is apparently no increase in the corrosion-fatigue limit with increase in size of specimen, have been conservatively stated.

In reply to Mr. Lessells' third question as to the relation between corrosion-fatigue with direct tension-compression range of stress, and corrosion-fatigue with repeated bending stresses, I would say that it seems probable that the relation between cyclic stress and the depth and sharpness of corrosion-pitting depends only on the cyclic stress range at the surface of the specimen, provided the diameter of the specimen is large in comparison with the depth of pitting. If this view is correct, the depth and sharpness of pitting should be practically the same, whether the applied range is direct tension-compression or repeated bending.

Mr. Lessells' question, therefore, becomes a question as to the relationship between the fatigue limits of equally notched specimens tested by direct tension-compression or by repeated bending. As the relationship between the fatigue limits of unnotched specimens tested in these two ways is still a subject of dispute, Mr. Lessells' question cannot be answered definitely. This reasoning would seem to indicate, however, that corrosion-fatigue by direct tension-compression would be nearly as damaging as by repeated bending. **Mr. McAdam.**

MR. SPELLER.—As an instance of the practical application of inhibitors, in rotary drilling of oil wells the outside and inside surface of the pipe is often continually exposed to rather corrosive mud-laden water, and it would be a very simple matter in that case to use an inhibitor such as we have described in such a situation when the fluid is limited in volume and is continuously recirculated. As a matter of fact, the upset threaded section of the drill pipe referred to by Mr. Fletcher is two and one half times the area of the thickness of the body of the pipe and failures do not now occur there as often as before. At least three-quarters of the failures now occur in the body of the pipe. **Mr. Speller.**

Referring to Mr. McAdam's criticism as to the method used in drawing our inhibitor-fatigue curve, we feel that the data at hand does not warrant the conclusion that the inhibitor gives a higher fatigue limit (due to cooling) than the air curve. Some further tests are under way with a higher concentration of inhibitor and the results may give a more definite answer on this point.

THE STATIC AND FATIGUE PROPERTIES OF SOME CAST IRONS

By J. B. Kommers[1]

Synopsis

The paper reports the results obtained in the testing of ten series of cast iron, each series comprising 31 bars cast in the form of the A.S.T.M. Standard Arbitration Bar. A complete history of the iron in each series is given with chemical analyses and metallographic properties.

The tests include tension tests on two different sizes of specimens, compression tests, impact tests, transverse tests, Rockwell and Brinell hardness tests and fatigue tests. The properties of two high-strength cast irons in the series are discussed and the effects of nickel and chromium additions are pointed out.

The several properties of the irons are compared; and it is shown that while the fatigue endurance limit of cast irons may be roughly estimated from properties such as tensile strength, hardness, and modulus of rupture, knowledge of the effect of available factors in influencing the properties of cast iron is so meager that such approximate estimates of endurance limit should always be checked by direct experiment. No consistent relation was found between endurance limit and chemical composition.

The paper suggests the need of further study along several lines such as influence of melting and pouring temperatures, heat treatment of castings and effect of alloy additions.

Introduction

About a year ago R. S. MacPherran, chairman of Committee A-3 on Cast Iron, approached the author in regard to some proposed tests of cast iron. Mr. MacPherran felt that cast iron had not received the attention that it deserved and that steps should be taken to increase the available information, keeping especially in view the problem of the production of cast irons of high strength.

Satisfactory arrangements were made whereby the expense of the investigation was borne partly by the cooperating manufacturers and partly by the University of Wisconsin. Manufacturers in different parts of the country and in various industries furnished standard arbitration bars, and the machining and testing of specimens was carried out at the University of Wisconsin. The work reported in this paper was made possible by the cooperation of the following manufacturers:

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[1] Professor of Mechanics, University of Wisconsin, Madison, Wis.

Allis-Chalmers Manufacturing Co., West Allis, Wis.
Buffalo Foundry and Machine Co., Buffalo, N. Y.
Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.
General Motors Corporation, Detroit, Mich.
Deere and Co., Moline, Ill.
Bucyrus-Erie Co., South Milwaukee, Wis.
Boynton Furnace Co., Jersey City, N. J.
American Cast Iron Pipe Co., Birmingham, Ala.

Acknowledgment is due the representatives of these manufacturing companies who were active in furnishing the material for the tests.

Acknowledgment is due John W. Bolton, of the Lunkenheimer Co. of Cincinnati, Ohio, who prepared the micrographs for this paper, and S. A. Weigand who assisted him.

Thanks are due especially to R. S. MacPherran and Hyman Bornstein, chairman and secretary, respectively, of Committee A-3 on Cast Iron, for their active interest and support in this work.

The work of G. S. Paul, student research assistant, in making static tests and checking computations, is gratefully acknowledged.

The present paper is a report on the results obtained on ten different cast irons, obtained from eight different manufacturers. The paper should be looked upon as a preliminary survey which it is hoped will help in pointing the way to further work which it may be desirable to carry on with cast iron.

MATERIALS, TESTS AND TEST SPECIMENS

In order that the reader might have rather full information regarding the material used in these tests, the data given in Table I is presented. This table gives information on the foundry mixtures used for the various cast irons, type of furnace used for melting the iron, pouring temperatures, kind of molds, and the use which the manufacturer makes of the particular cast iron reported.

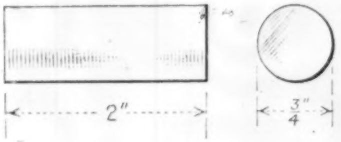
The material was cast in the form of the A.S.T.M. standard arbitration bar, and each manufacturer supplied thirty-one bars for each cast iron investigated. The bars were marked with a letter for each series and numbers from one to thirty-one. The specimens cut from the bars were marked to correspond.

Table II shows the chemical compositions of the various cast irons. Each value in the table is the average of two determinations made at different times on two different samples, and in all but four cases, by two different laboratories.

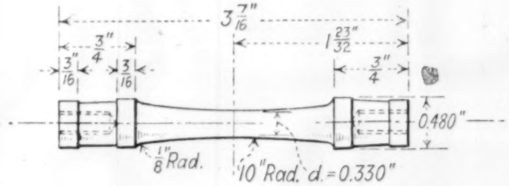
TABLE I.—MANUFACTURING DATA ON TEN CAST IRONS USED IN INVESTIGATION.

Series	Kind of Iron	Mixtures	Pouring Temperature, deg. Fahr.	Molds	Use Made of Iron
A	Cupola	25 per cent No. 3 Northern Charcoal Pig Iron. 37.5 per cent Northern Coke Pig Iron. 12.5 per cent Foundry Scrap. 25 per cent Steel Rails.	2300	Used cores, without blacking. Two bars per mold.	Cylinders, gears, crusher castings, and in general large castings requiring strength.
B	Cupola	35 per cent Foundry Pig Iron 1.75 to 2.25 per cent Silicon. 40 per cent Shop Scrap. 23 per cent Steel. 2 per cent Spiegeleisen, 19 to 21 per cent Manganese.	2650	Dried core, silica core sand, 95 to 99 per cent Silica. Foot's linseed oil, graphite blacking. One bar per mold.	Steam drums, 32 in. in diameter by 72 in. in length, of about 4000 lb. weight.
C	Cupola	63 per cent Southern Pig Iron. 20 per cent Return Scrap. 17 per cent Steel Rails.	2400 to 2550	Clay-sand core, blacked and dried. One bar per mold.	Iron pipe, centrifugally cast, 4 to 12 in. in size.
D	Air Furnace	23 per cent No. 1 Foundry Pig Iron. 12 per cent No. 3 Foundry Pig Iron. 60 per cent Cast-Iron Scrap. 5 per cent Silicon Pig Iron, 8 per cent Silicon.	2300	Used A.S.T.M. recommendations, except all bars were made up in one flask.	Electric generator frames, bed plates, spiders, steam turbines, condenser shells, large pumps.
E	Cupola	20 per cent Foundry Pig Iron, 2.10 per cent Silicon. 30 per cent Malleable Pig Iron, 1.80 per cent Silicon. 30 per cent Cast Iron Scrap. 12 per cent Steel. 3 per cent Malleable Scrap. 5 per cent Silvery Iron, 16 per cent Silicon.	2400 to 2500	Green sand mold. Two bars per mold.	Transmission case, manifolds, housings, covers, and other automobile motor parts.
F	Cupola	45 per cent Pig Iron. 45 per cent Cast Iron Scrap. 10 per cent Steel Scrap.	2500	Oil sand core, silica sand, no blacking. Four bars per mold.	Tractor cylinders.
G	Cupola	45 per cent Pig Iron. 45 per cent Cast Iron Scrap. 10 per cent Steel Scrap.	2500	Oil sand core, silica sand, no blacking. Four bars per mold.	Tractor castings, excepting cylinders.
H	Cupola	93 per cent Steel Scrap. 4.5 per cent Ferro-silicon (50-per-cent). 2.5 per cent Ferro-manganese (80-per-cent).	Not known. Hotter than for ordinary iron.	Cores, black sand, no oil or blacking. Two bars per mold.	Rope drums, gear cases, and miscellaneous high-strength castings.
J	Cupola	30 per cent No. 2-X Pig Iron. 36.67 per cent No. 2 Pig Iron. 33.33 per cent Cast Iron Scrap.	2490	Green sand molds, no blacking. Four bars per mold.	Coal ranges, hot air furnaces, hot water supply boilers, hot water and steam heating boilers.
K	Cupola	93 per cent Steel Scrap. 4.5 per cent Ferro-silicon (50-per-cent). 2.5 per cent Ferro-manganese (80-per-cent).	Not known. Hotter than for ordinary iron.	Cores, black sand, no oil or blacking. Two bars per mold.	Special cast, not a regular product.

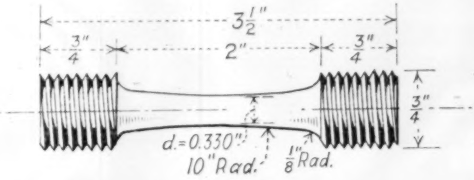
In preparing specimens, ½ to 1 in. was cut off the large end of the bars, and the fatigue and other specimens were then cut off. Fifteen bars were set aside for fatigue specimens, one specimen being cut from the larger end of each bar. When more than fifteen fatigue specimens were wanted, a second specimen was cut off. Six additional bars were used in the same way for the six tension test specimens. This left ten bars, five of which were used for the transverse test and five for the impact test. For the latter two tests the full-size arbitration bars were used.



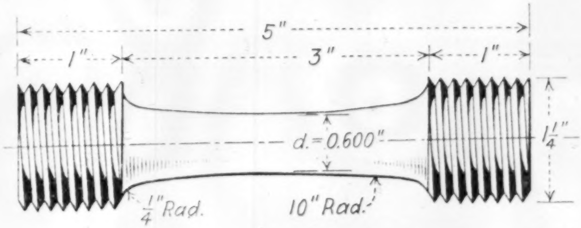
Compression Specimen.


3 7/16"
1 23/32"
3/4"
3/4"
3/16"
3/16"
0.480"
1/8" Rad.
10" Rad.
d.=0.330"

Fatigue Specimen.


3 1/2"
3/4"
2"
3/4"
3/4"
d.=0.330"
10" Rad.
1/8" Rad.

Tension Specimen.


5"
1"
3"
1"
d.=0.600"
1 1/4"
1/4" Rad.
10" Rad.

Tension Specimen.

FIG. 1.—Compression, Fatigue, and Tension Specimens.

The tests which were made on the various cast irons were tension, compression, transverse, impact, Brinell hardness, Rockwell hardness and fatigue. Micrographs were made showing the structure before etching, and after etching with picric acid.

Figure 1 shows the tension, compression and fatigue test specimens which were employed. It will be noted that two different sizes of tension test specimens were used. The reason for this was to have one specimen which would have the same minimum diameter as the fatigue specimen, and a larger one which would represent better the average strength of the arbitration bar. At the same time the tensile results on the two sizes would indicate whether there was much difference in strength in going from the center of the bar to the outside.

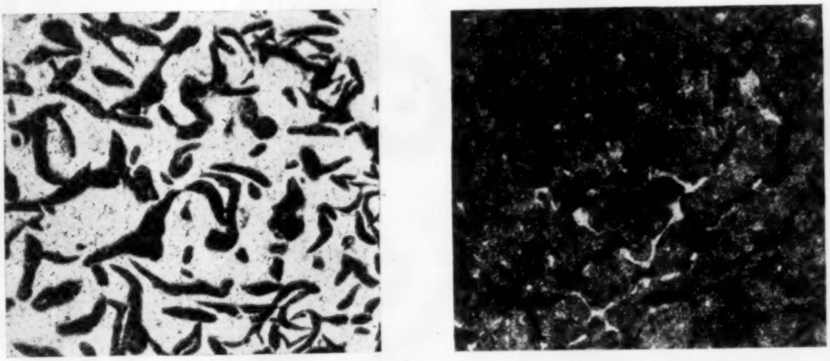
The shape of the tension test specimen was dictated by the same considerations that governed the shape of the fatigue specimen, that is, the desire to prevent localized stress due to shoulders and other abrupt changes in cross-section, which must especially be avoided in a material as brittle as cast iron. Out of sixty tension specimens only two, of the smaller size, broke at the shoulder and these failures were due to flaws. In these cases the results were discarded and new specimens were made and tested. The evidence of these tests, therefore, indicates that the design chosen for the tension specimens was a satisfactory one to accomplish the purpose intended.

The transverse tests were made on a small foundry tester, which supported the specimen on a span of 12 in. and loaded it at the center. Deflections of the specimen were measured at the center of the span.

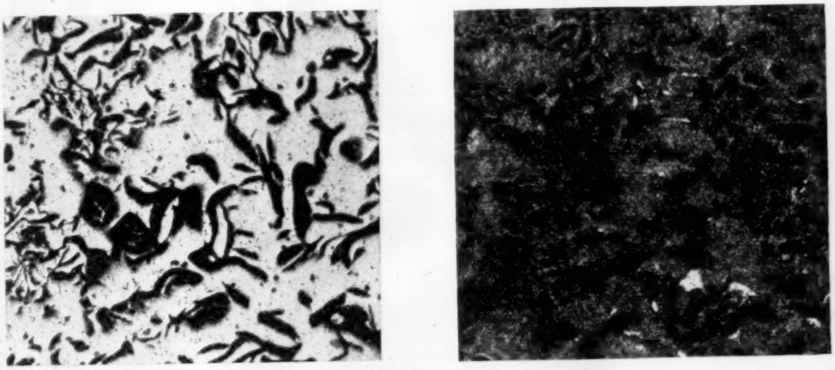
The impact tests were made on a Russell impact machine, using a 12-in. span. The Russell machine consists essentially of a slab of steel weighing 185 lb. which can rotate as a pendulum about an axis at one end. By means of suitable dials, the drop of the center of gravity of the slab before striking the specimen and the rise after breaking the specimen may be measured and thus the energy of rupture calculated.

From five of the bars from which a fatigue specimen had been cut, three slugs about $\frac{1}{2}$ in. thick were cut from each bar, making fifteen slugs in all. One set of five was used for the Brinell test, one set for the Rockwell test, and one set for metallographic examination.

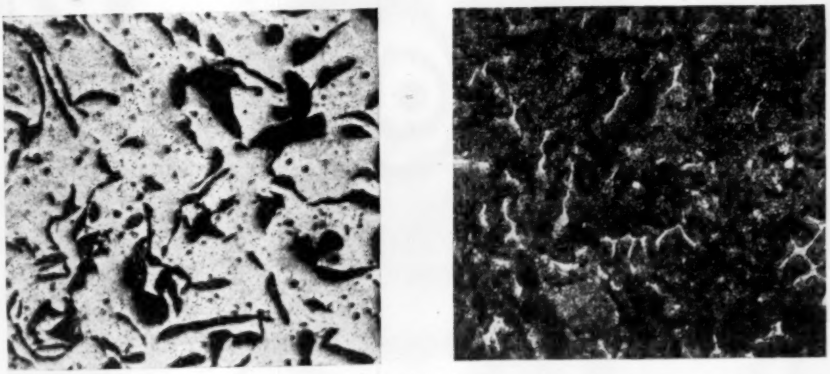
In preparing the fatigue specimens, the specimens were first rough-turned and then ground to size. For polishing the surface, French emery papers No. 1 and No. 00 were first used, after which a mixture of oil and optical emery powder No. 300 was used on the worn emery papers. The last operation consisted of buffing the specimens on a cloth buffing wheel, transversely and longitudinally, using a commercial metal polish. The fatigue tests were carried out in R. R. Moore machines, which have been described in previous papers before this



Series A



Series B

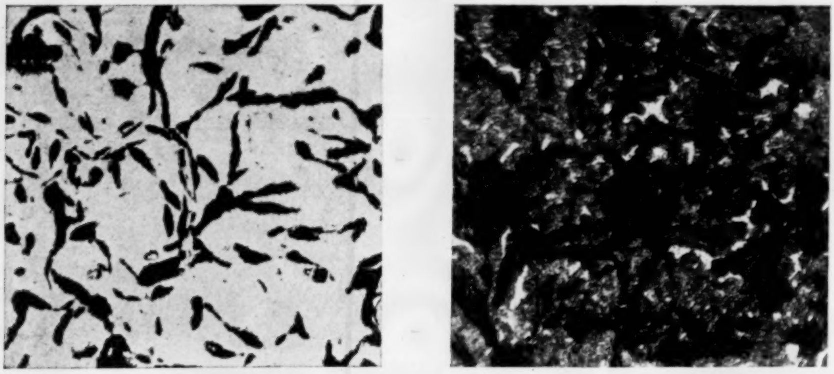


Series C

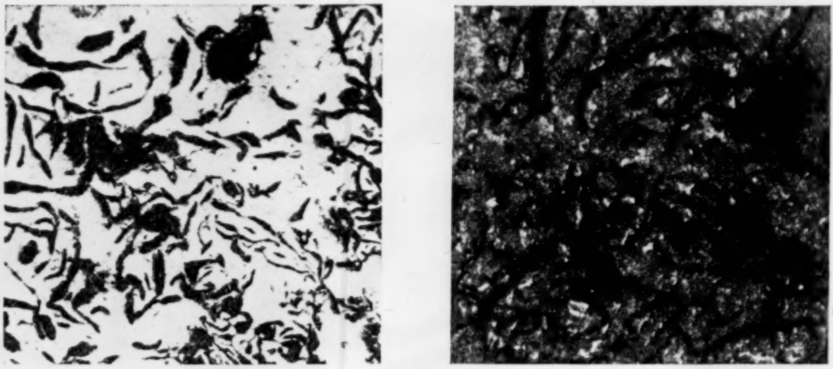
Unetched

Etched

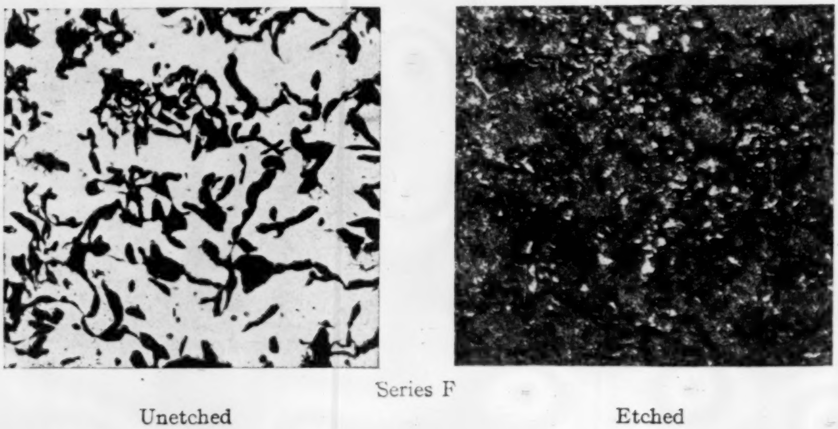
FIG. 2.—Micrographs of Cast Iron of Series A, B, and C (×100).



Series D



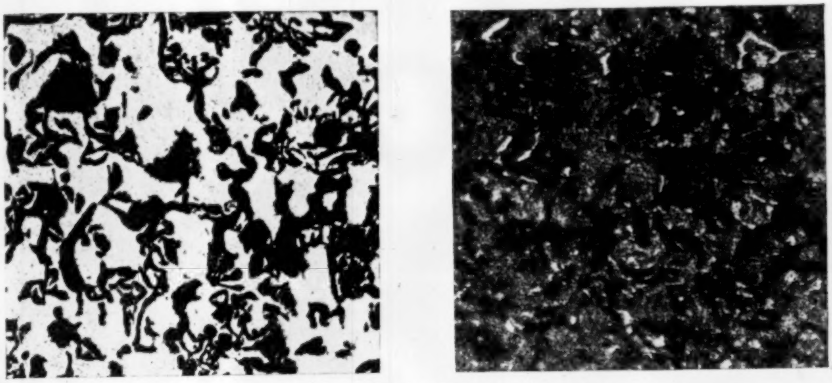
Series E



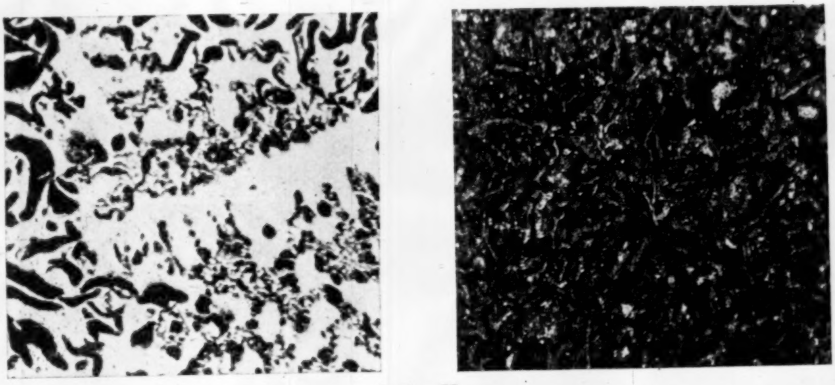
Series F

Unetched Etched

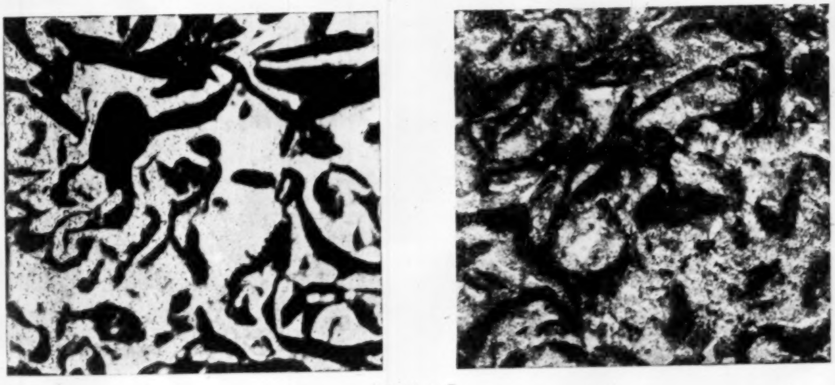
FIG. 3.—Micrographs of Cast Iron of Series D, E, and F (×100).



Series G



Series H



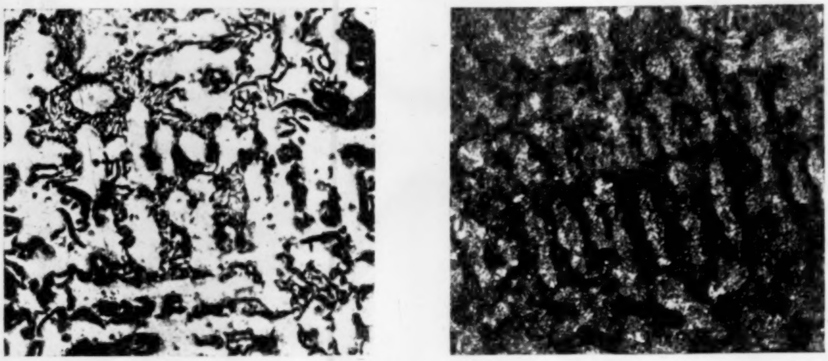
Series J

Unetched Etched

FIG. 4.—Micrographs of Cast Iron of Series G, H, and J (×100)

Society.[1] The specimens were subjected to completely reversed bending stresses of tension and compression, at a rate of about 1750 r.p.m. In general, 20,000,000 cycles were used as the maximum run; that is, if the specimen had not failed after being subjected to 20,000,000 cycles, it was assumed that the stress was at or below the endurance limit. The *S-N* diagrams, referred to later, show that a run of 20,000,000 cycles is ample in determining the endurance limit.

Figures 2, 3, 4, and 5 show the micrographs for the various cast irons. The author is indebted to J. W. Bolton for the description of these structures given below. It should be stated here that series *F* and *G* were alike except for the presence of nickel and chromium in series *F*. Series *H* and *K* were also supposed to be alike except for the presence of nickel in series *H*.



Series K

Unetched

Etched

FIG. 5.—Micrographs of Cast Iron of Series K (×100).

Series *A*, although containing only 1.12 per cent of silicon, shows rather coarse graphite flakes, being mostly of the type termed "secondary graphite." This indicates (1) rather slow cooling in comparison to what is usually found for this analysis; or (2) the presence of residual graphite in the liquidus. Although the matrix approaches the pearlitic (as shown both by the etched sample and by analysis), the metal is only of medium strength. This is a good illustration of the fact that the graphite flakes themselves have much to do with the strength of the metal. The amount of steadite is small and at higher magnifications was found to be structureless.

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[1] R. R. Moore, "Effect of Grooves, Threads, and Corrosion upon the Fatigue of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 255 (1926); and "Effect of Corrosion upon the Fatigue Resistance of Thin Duralumin," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 128 (1927).

Series *B* evidently cooled a little faster than series *A* or else had less residual graphite in the melt, for there is an appreciable amount of primary graphite. The etched sample shows a matrix approaching the pearlitic, with a small amount of structureless steadite.

Series *C* has coarser graphite, with considerable steadite in the matrix. Although not apparent at the low magnification, cellular structure was apparent at higher magnifications.

Series *D* has fairly coarse graphite, but apparently not quite so much of it. The lower magnification does not reveal much free ferrite.

Series *E* is a peculiar sample. One would naturally expect a considerable amount of free ferrite, but micro-examination shows that this separation has just begun. The graphite is part primary, part secondary. as usually found in irons giving a tensile strength of 30,000 lb. per sq. in.

TABLE II.—CHEMICAL COMPOSITION OF CAST IRONS.

Each value is average of two determinations made at different times on two different samples.

Series	Total Carbon, per cent	Graphitic Carbon, per cent	Combined Carbon, per cent	Silicon, per cent	Sulfur, per cent	Phosphorus, per cent	Manganese, per cent	Nickel, per cent	Chromium, per cent	Vanadium, per cent	Titanium, per cent
A	3.30	2.66	0.64	1.12	0.090	0.21	0.62	....	....	....	....
B	3.31	2.70	0.61	1.45	0.106	0.367	0.86	....	....	....	....
C	3.51	2.82	0.69	1.58	0.088	0.72	0.54	0.00	0.01	0.05	0.06
D	3.23	2.66	0.57	1.96	0.072	0.44	0.54	....	....	....	....
E	3.24	2.84	0.40	2.85	0.101	0.168	0.57	....	....	....	....
F	3.46	3.02	0.44	2.35	0.078	0.27	0.62	0.63	0.22	....	....
G	3.35	2.85	0.50	2.25	0.084	0.26	0.60	....	....	....	....
H	2.98	2.29	0.69	2.22	0.082	0.057	0.52	1.15	....	....	....
J	3.68	3.13	0.55	2.17	0.08	0.646	0.54	....	....	....	....
K	2.86	2.21	0.65	2.51	0.079	0.057	0.62	....	....	....	....

Series *F* evidently had considerable graphite growth. The rosette formation is often found in irons of about 3.50 per cent of carbon and high silicon. The etched sample shows considerable ferrite, particularly in the interstices of the rosette or whorl formation. Steadite is low and structureless.

Series *G* probably had a somewhat more favorable thermal history than series *F*. The ferrite is not so apparent. The expected strength might be somewhat greater than for series *F*.

Series *H* and *K* are somewhat alike; series *H* has slightly more primary graphite, although the secondary graphite is also slightly larger in series *H*. In both series the primary graphite has been rejected to the boundaries of the crystals of primary austenite. The presence of these primary austenite dendrites indicates a metal rather low in total carbon. Both series are nearly pearlitic and practically no steadite is visible.

TABLE III.—SUMMARY OF RESULTS OF TESTS.

	Tensile Strength		Compression Test		Hardness Test			Transverse Test			Impact	Fatigue
	Small Specimen, lb. per sq. in.	Large Specimen, lb. per sq. in.	Unit Stress at First Bend, lb. per sq. in.	Unit Stress at Ultimate, lb. per sq. in.	Brinell Number	Rockwell Number, "B" Scale		Maximum Load, lb.	Maximum Deflection, in.	Modulus of Rupture, lb. per sq. in.	Energy of Rupture, in-lb. per cu. in.	Endurance Limit, lb. per sq. in.
						At Center	At Mid-radius					
Series A.	34 900	28 300	77 200	109 200	197.0	90.0	95.0	5030	0.162	78 700	77.3[a]	
	34 600	39 300	76 200	107 900	201.0	87.5	93.0	4630	0.129	72 700	44.2	
	37 500	37 600	70 000	109 800	203.0	89.0	94.0	4750	0.159	74 100	38.9	
	.....	.....	.....	......	203.0	85.0	93.0	4730	0.156	74 600	34.4	
	.....	.....	.....	......	207.0	90.0	93.0	4430	0.140	69 200	31.0	
Average...	35 700	35 100	74 500	109 000	202.0	88.3	93.6	4714	0.149	73 900	37.1	19 000
Series B.	38 700	42 000	73 600	138 700	207.0	96.5	98.3	4280	0.118	67 000	22.5	
	40 000	39 600	78 100	138 000	207.0	98.0	99.0	4110	0.112	64 300	16.4	
	39 800	37 400	81 500	136 100	212.0	95.8	98.0	4040	0.111	63 300	19.8	
	.....	.....	.....	......	201.0	93.0	98.0	4200	0.111	65 800	19.2	
	.....	.....	.....	......	209.5	98.5	99.0	4220	0.115	66 300	27.8	
Average...	39 500	39 700	77 700	137 600	207.3	96.3	98.5	4170	0.113	65 300	21.1	19 000
Series C.	27 500	31 400	81 200	117 800	197.0	90.5	97.5	3880	0.124	60 800	17.2	
	32 700	32 600	75 700	114 500	192.0	90.0	98.0	3560	0.097	55 800	17.0	
	30 800	33 000	80 400	120 400	197.0	88.5	96.0	3720	0.111	58 400	17.0	
	.....	.....	.....	......	189.5	87.5	96.0	3720	0.113	58 300	22.5	
	.....	.....	.....	......	192.0	92.0	95.0	3640	0.112	56 800	17.5	
Average...	30 300	32 300	79 100	117 600	193.5	89.7	96.5	3704	0.111	58 000	18.2	15 000
Series D.	33 100	31 700	83 300	123 800	194.5	93.0	94.0	4030	0.127	63 200	23.6	
	33 000	28 800	83 100	121 600	192.0	88.0	96.0	3740	0.100	56 200	22.9	
	29 600	30 600	81 500	123 000	192.0	93.0	97.0	3580	0.120	58 300	21.6	
	.....	.....	.....	......	187.0	88.0	97.0	3800	0.111	59 300	21.6	
	.....	.....	.....	......	197.0	84.0	96.0	3960	0.125	62 000	18.6	
Average...	31 900	30 400	82 600	122 800	192.5	89.2	96.0	3822	0.117	59 800	21.7	12 000
Series E.	32 500	30 600	77 700	116 900	185.8	91.5	97.0	3520	0.122	55 300	24.4	
	32 100	31 500	85 600	118 800	187.0	94.5	96.0	3760	0.126	58 700	23.3	
	31 200	31 100	74 300	115 400	179.0	93.5	96.0	3960	0.140	62 300	26.5	
	.....	.....	.....	......	183.0	96.0	96.0	4000	0.145	62 700	20.6	
	.....	.....	.....	......	187.0	92.5	95.0	3900	0.138	61 300	22.2	
Average...	31 900	31 100	79 200	117 000	184.4	93.6	96.0	3830	0.134	60 100	23.4	13 400
Series F.	38 600	35 700	82 700	133 200	209.5	95.5	97.5	4600	0.153	72 500	30.6	
	35 400	36 400	82 800	129 100	214.5	97.5	97.5	4260	0.133	66 800	25.1	
	37 200	34 600	80 900	133 700	209.5	95.0	97.5	4670	0.148	73 500	27.8	
	.....	.....	.....	......	207.0	97.0	98.0	4080	0.121	64 000	17.3	
	.....	.....	.....	......	207.0	94.0	97.5	4120	0.112	64 400	23.8	
Average...	37 100	35 600	82 100	132 000	209.5	95.8	97.6	4346	0.133	68 200	24.9	19 500
Series G.	36 700	33 400	76 300	128 700	194.5	88.0	93.0	4360	0.140	68 600	18.4	
	37 300	33 000	77 500	135 900	198.5	95.0	96.5	4660	0.171	73 200	30.7	
	32 900	34 800	78 800	128 200	197.0	92.0	96.5	3760	0.111	59 000	34.9	
	.....	.....	.....	......	197.0	96.0	96.0	4620	0.168	72 300	23.6	
	.....	.....	.....	......	201.0	93.0	96.0	4490	0.168	70 200	26.4	
Average...	35 600	33 700	77 500	130 900	197.6	92.8	95.6	4378	0.152	68 700	26.8	19 100
Series H.	55 500	51 500	83 300	158 500	241.0	100.5	101.0	5720	0.180	89 800	45.1	
	43 200	48 400	83 600	145 000	248.0	100.0	100.0	5460	0.162	85 500	41.6	
	54 200	44 300	81 100	165 300	235.0	99.0	101.0	5530	0.185	86 800	48.1	
	.....	.....	.....	......	237.0	100.0	100.0	5660	0.190	88 700	43.6	
	.....	.....	.....	......	233.0	102.0	101.0	5770	0.189	90 400	41.5	
Average...	51 000	48 100	82 700	156 300	238.8	100.3	100.6	5628	0.181	88 200	44.0	22 000

[a] Not included in determining the average.

NOTE.—The loads in the transverse test have been reduced to a common diameter of 1.25 in.

TABLE III.—SUMMARY OF RESULTS OF TESTS—*Continued.*

	Tensile Strength		Compression Test		Hardness Test			Transverse Test			Impact	Fatigue
						Rockwell Number, "B" Scale						
	Small Specimen, lb. per sq. in.	Large Specimen, lb. per sq. in.	Unit Stress at First Bend, lb. per sq. in.	Unit Stress at Ultimate, lb. per sq. in.	Brinell Number	At Center	At Mid-radius	Maximum Load, lb.	Maximum Deflection, in.	Modulus of Rupture, lb. per sq. in.	Energy of Rupture, in lb. per cu. in.	Endurance Limit, lb. per sq. in.
Series J.	23 600	23 600	74 000	86 000	179.0	86.5	88.0	3260	0.113	51 100	15.9	
	26 000	24 700	73 000	85 000	174.0	87.0	89.0	3690	0.141	57 700	16.7	
	20 000	26 500	74 000	84 000	179.0	90.0	88.5	3520	0.127	55 100	20.0	
	.....	.....	.....	......	174.0	86.5	89.5	3200	0.102	50 100	17.7	
	.....	.....	.....	......	152.0	84.0	85.5	3580	0.134	56 100	12.4	
Average...	23 200	24 900	73 700	85 000	171.6	86.8	88.1	3450	0.123	54 000	16.5	11 800
Series K.	41 900	47 600	79 200	152 800	241.0	92.5	99.0	4830	0.132	75 500	36.0	
	40 700	52 300	75 600	152 600	229.0	95.5	99.0	5390	0.125	84 300	28.6	
	45 100	51 000	82 000	158 800	235.0	96.5	98.5	4840	0.121	76 000	....	
	.....	.....	.....	......	244.5	98.5	99.0	4340	0.100	67 400	....	
	.....	.....	.....	......	235.0	97.5	98.5	4780	0.119	74 800	....	
Average...	42 500	50 300	78 900	154 700	236.9	96.1	98.8	4836	0.119	75 600	32.3	24 100

NOTE—The loads in the transverse test have been reduced to a common diameter of 1.25 in.

Series *J* is a weak, open-grain type, with practically all secondary graphite. Some decomposition of pearlite is evident and considerable steadite is present.

TEST DATA AND RESULTS

Table III gives a summary of the tension, compression, hardness, transverse, impact and fatigue tests. The individual results have been shown for the tension, compression, hardness, transverse and impact tests, so that an idea of the uniformity of the material may be obtained.

For series *A* the impact result marked with a superior "*a*" was not included in determining the average. This specimen did not break at the center of the span as did the other bars and undoubtedly is not representative of the material.

Figures 6, 7, and 8 show the *S–N* diagrams for the various cast irons as determined from the fatigue tests. In these diagrams the value of unit stress, *S*, to which the specimen was subjected is plotted as the ordinate, and the value of number of cycles for rupture, *N*, is plotted to logarithmic scale as the abscissa.

Figures 9, 10, and 11 show the correlation between endurance limit and the various results obtained from the other tests.

Figures 12, 13, 14 and 15 show the correlation between the various test results, not including the fatigue tests. What seemed to be the more important relations between the various results have been plotted.

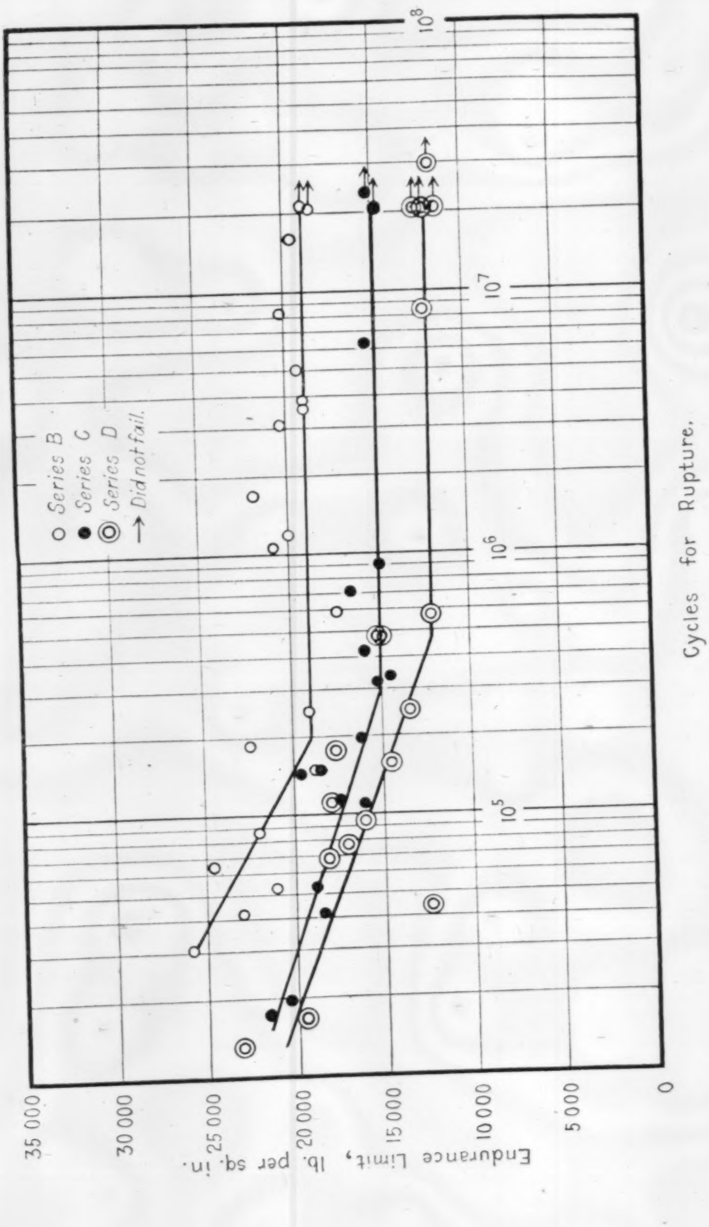

FIG. 6.—*S* - *N* Diagram for Series B, C, and D.

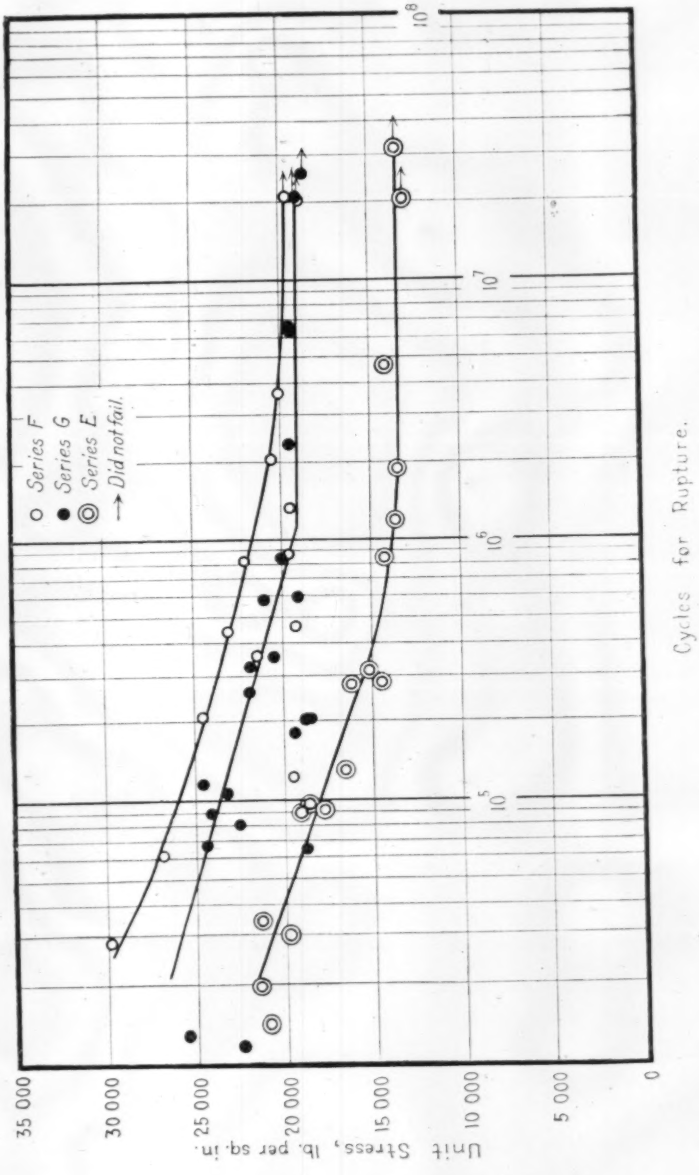

FIG. 7.—*S* – *N* Diagram for Series F, G, and E.

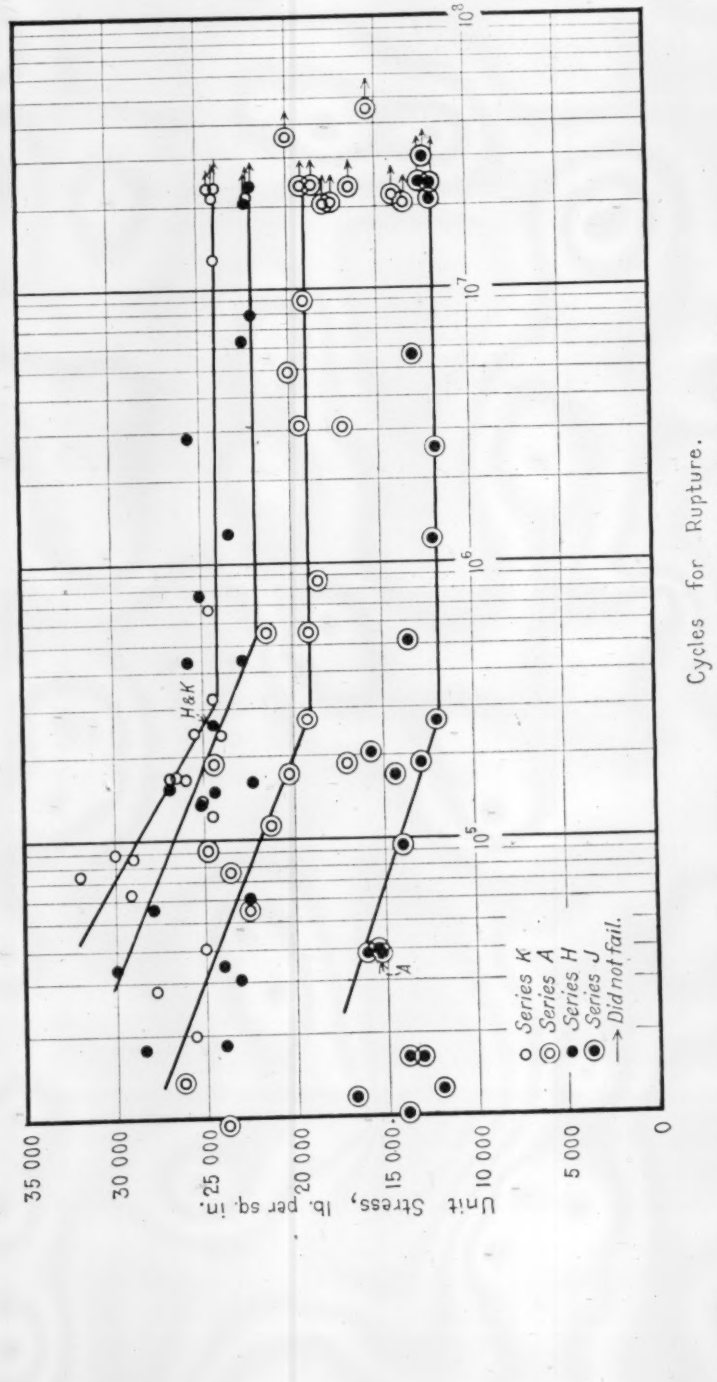

FIG. 8.—*S* - *N* Diagram for Series K, A, H, and J.

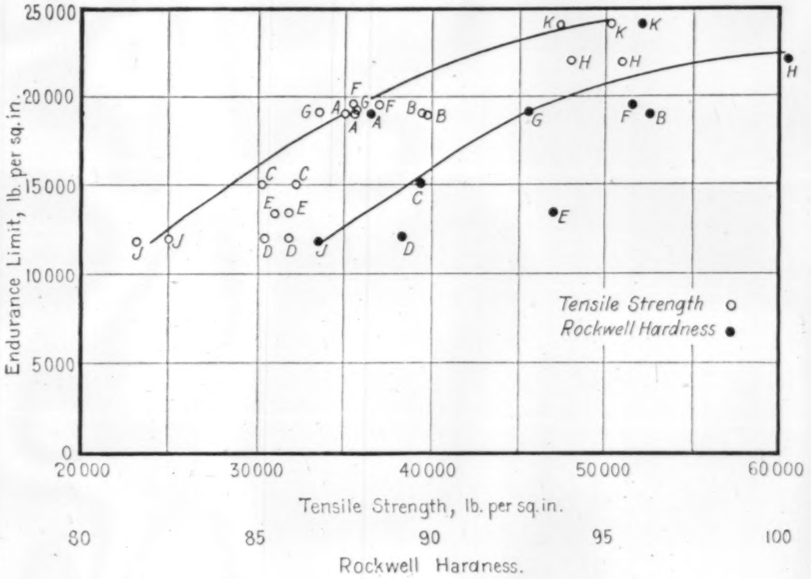

FIG. 9.—Relation Between Endurance Limit and Tensile Strength, and Endurance Limit and Rockwell Hardness.

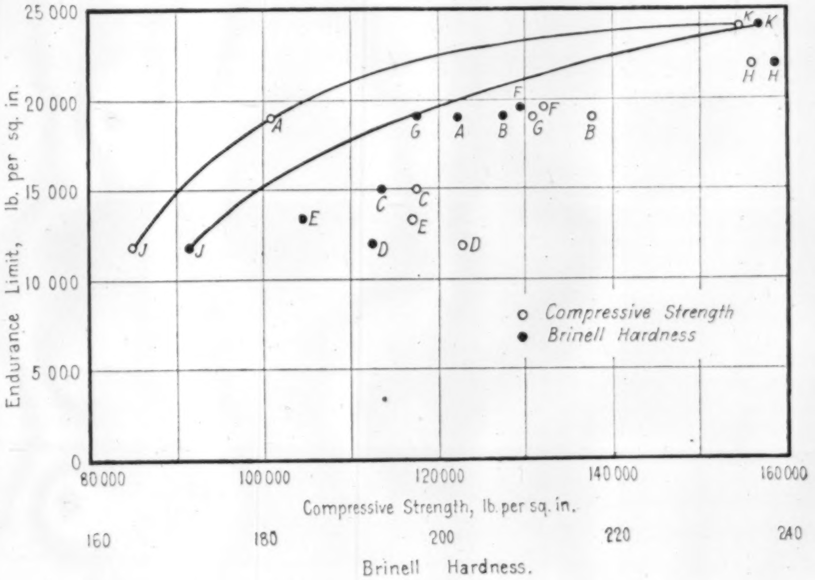

FIG. 10.—Relation Between Endurance Limit and Compressive Strength, and Endurance Limit and Brinell Hardness.

DISCUSSION OF RESULTS

Uniformity of Material:

In general the results of the hardness, compression and transverse tests are fairly uniform for any given series, and this would indicate that the material was fairly uniform. This is not true of the tensile, impact and fatigue results. These latter three tests seem to be much

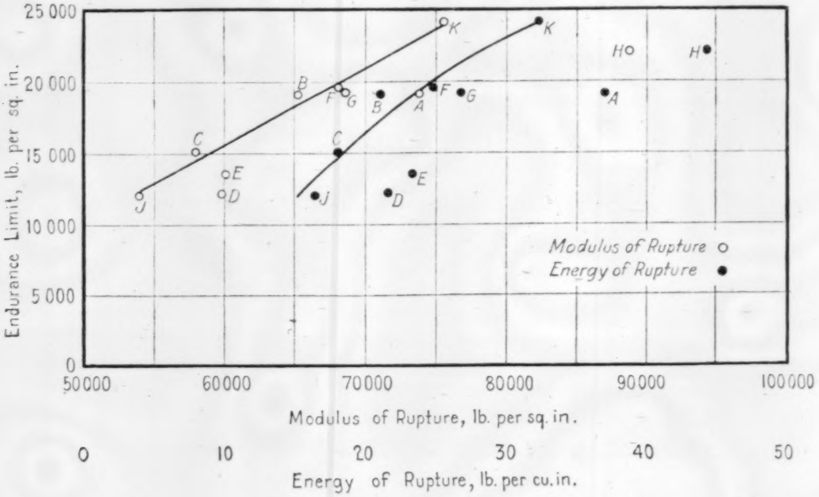

FIG. 11.—Relation Between Endurance Limit and Modulus of Rupture, and Endurance Limit and Energy of Rupture.

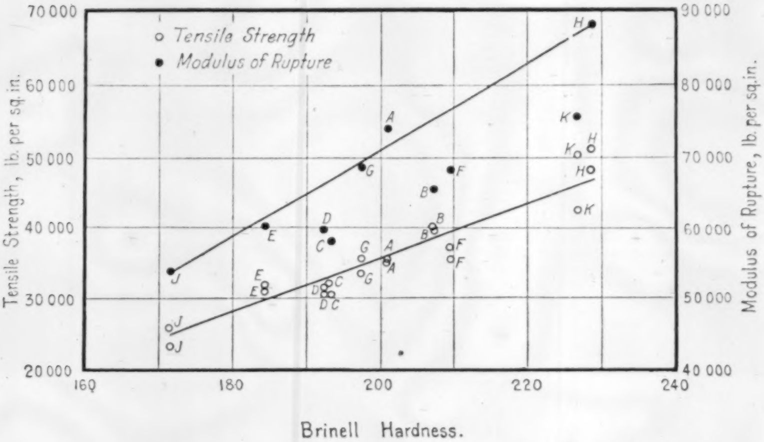

FIG. 12.—Relation Between Brinell Hardness and Tensile Strength, and Brinell Hardness and Modulus of Rupture.

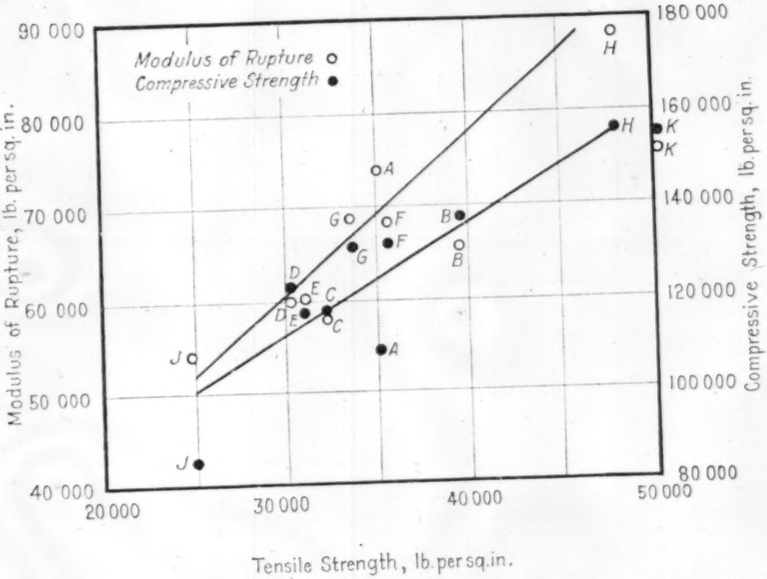

FIG. 13.—Relation Between Tensile Strength and Modulus of Rupture, and Tensile Strength and Compressive Strength.

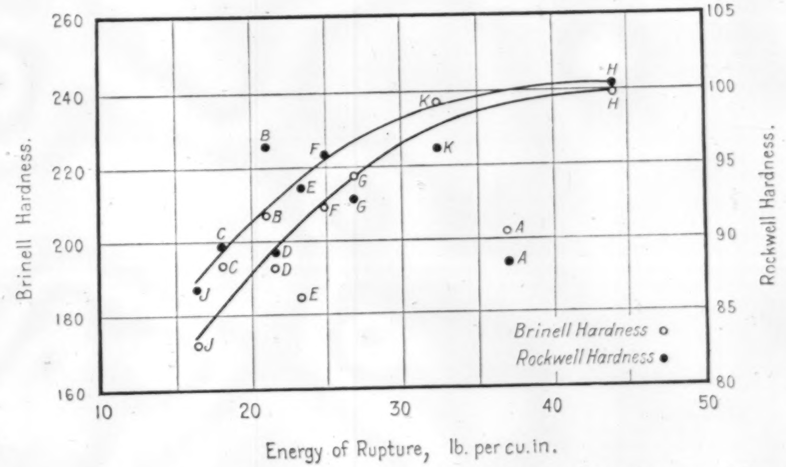

FIG. 14.—Relation Between Energy of Rupture and Brinell Hardness, and Energy of Rupture and Rockwell Hardness.

more sensitive to differences in specimens, and presumably are much more sensitive in picking out flaws and weaknesses if they exist.

It will be noted that in general the *S–N* diagrams for the fatigue results show considerable scatter, much more than is encountered with uniform ferrous or non-ferrous materials. This would indicate that cast iron in general is not as homogeneous as some of these other materials. Such a conclusion would of course be expected.

The *S–N* diagram for series *E* is probably the most uniform of all the cast irons tested. A glance at Table III will show that this uniformity is also indicated by the results on this series recorded in the table. Series *A*, which shows considerable scatter on the *S–N* diagram for the fatigue results, also shows considerable variation for the tensile and impact results. Series *B*, which is somewhat more uniform than series *A* on the *S–N* diagram, also shows better uniformity than series *A* for the tensile results but not for the impact.

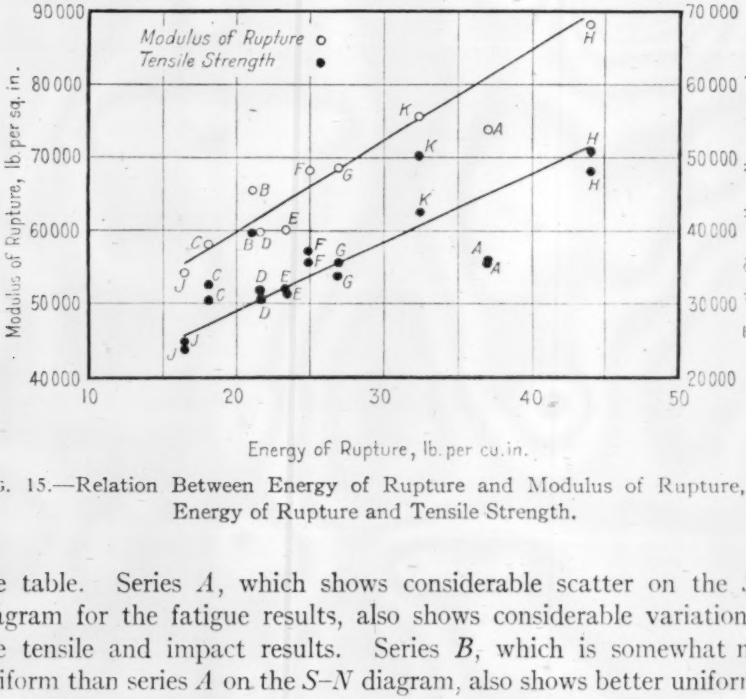

FIG. 15.—Relation Between Energy of Rupture and Modulus of Rupture, and Energy of Rupture and Tensile Strength.

Series *F* and *G*, which show considerable variation on the *S–N* diagrams, also show considerable variation for the impact results, but are quite uniform for the tensile results. Series *C* and *D*, which are fairly uniform on the *S–N* diagrams, are also quite uniform for the tensile and impact values. Series *H*, which shows more scatter on the *S–N* diagram than does series *K*, also shows somewhat more variation in the tensile results. Series *J* shows some scatter on the *S–N* diagram, and also some variation for the impact results.

In general the conclusion may be drawn from these results that the tensile, impact and fatigue tests are more sensitive to differences in material than the hardness, compression and transverse tests.

A characteristic result which shows non-uniformity of material was indicated by a number of the irons, for instance series *F*, *G*, *H*, *J* and *K*. On the *S–N* diagram for all of these will be found plotted points near the left edge of the diagram, lying lower than the average curve. This indicates that quite a number of specimens did not show the endurance for a given unit stress which might be expected from a normal sample. Such abnormality was strikingly brought out by three specimens of series *J* which showed numerous small pits over the entire surface even after they were polished. These specimens when tested all showed abnormally low endurance for a given unit stress. These specimens were so obviously abnormal that the results were not plotted on the *S–N* diagram.

High-Strength Cast Iron:

Series *H* and *K* were supposed to be alike except for the addition of 1.15 per cent of nickel to series *H*. These two irons are considerably stronger than any of the others and may be spoken of as high-strength cast irons. The endurance limits found for these two cast irons are the highest values for cast iron thus far reported, as far as the author is aware. The highest value previously reported was found by C. H. Bulleid[1] and was 21,700 lb. per sq. in. The fact that series *J*, which gave the lowest value of the present series, had an endurance limit of 11,800 lb. per sq. in. and that series *K* gave a value of 24,100 lb. per sq. in., indicates that cast iron need not be looked upon as a uniformly low-strength material. The strength of cast iron can evidently be controlled in a manner similar at least to that in the case of steel.

Effect of Nickel and Chromium:

Series *F* and *G* were the same in composition except that series *F* had received an addition in the ladle of 0.66 per cent of nickel and 0.25 per cent of chromium. Table III shows that the addition of these alloys had a slight strengthening effect. As previously stated, series *H* and *K* were supposed to be the same in composition except that series *H* had added to it 1.15 per cent of nickel. Most of the values in Table III are higher for series *H* except the endurance limit, which is higher for series *K*. Series *H* shows a considerable advantage in the modulus of rupture and energy of rupture.

While the above results, especially series *F* and *G*, show no outstanding advantage for the alloy irons, it must be remembered that the tests here used may not be such as to bring out the advantages of the use of alloys. One important use of alloys is to produce uniform hardness in castings which vary in thickness, and the present tests are not such as will throw light on this question.

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[1] *Engineering* (London), October 1, 1926, p. 429.

Hardness Tests:

It may be worth noting that on the Rockwell scale the softest iron gave a result of 88.1 and the hardest a result of 100.6, a difference of 12.5 points. The Brinell test gave a value of 171.6 for the softest iron and 238.8 for the hardest, a difference of 67.2 points. It may be concluded from these results that the Brinell scale of hardness would be more likely to point out slight differences in hardness than the Rockwell scale.

Correlation of Endurance Limit with Other Results:

With the results of ten different cast irons available, it seemed desirable to determine whether any correlation existed between endurance limit and the results from the other tests. Figures 9, 10, and 11 show these relations.

In Fig. 9 the endurance limit has been plotted as the ordinate and the tensile strength and Rockwell hardness as abscissa. The tensile results for both the small and large specimens were plotted, since Table III shows that there is very little difference in these results. In Fig. 9 no attempt was made to draw average curves through the plotted points, but instead the curves were drawn through the points showing the highest values of endurance limit. This shows the general relation between the quantities and at the same time indicates which cast irons showed a particularly high endurance limit for given values of tensile strength or hardness. The various iron series may be identified by the letter placed next to each plotted point.

The curves show that the endurance limit increases with increase of tensile strength and hardness, but the endurance limit tends to approach an asymptotic value at the higher values of strength and hardness.

Figure 10 shows the correlation between endurance limit and compressive strength, and also Brinell hardness. The curves show the same general tendency as in Fig. 9. In both Figs. 9 and 10 it would be interesting to know why some of the irons show such low values of endurance limit considering their strength and hardness.

Figure 11 shows the correlation between endurance limit and modulus of rupture, and also energy of rupture. If the upper ends of the curves had been drawn through the points *H* instead of *K* the curves would again show a marked falling off of endurance limit at high values of modulus of rupture and energy of rupture.

These results indicate that an approximate value of the endurance limit may be estimated from any of these curves. The simplest test would probably be to determine the Brinell hardness and estimate the endurance limit from the endurance-limit-hardness curve. At the present time, however, the information of the effect of variable factors in influencing the properties of cast iron is so meager that such approximate determinations should always be checked by direct experiment.

In Table IV the correlation between endurance limit and the other test results has been indicated by computing for each series the ratio of the endurance limit to tensile strength, and in the same way the ratio of endurance limit to the other test results. It will be noted that the values of any given ratio are not constant but vary somewhat. The extent of this variation has been shown by calculating the average ratio, and then computing the mean variation from the average in per cent. These computations show that the best correlation is between endurance limit and tensile strength, compressive strength and modulus of rupture. For these three strength values the mean variation from the average for these ratios is about 10 or 11 per cent.

TABLE IV.—CORRELATION BETWEEN ENDURANCE LIMIT AND OTHER TEST RESULTS.

	Ratio, Endurance Limit to Tensile Strength	Ratio, Endurance Limit to Compressive Strength	Ratio, Endurance Limit to Rockwell Hardness	Ratio, Endurance Limit to Brinell Hardness	Ratio, Endurance Limit to Modulus of Rupture	Ratio, Endurance Limit to Energy of Rupture
Series A	0.53	0.174	216	94.0	0.26	512
Series B	0.48	0.138	197	91.8	0.29	900
Series C	0.50	0.128	168	77.6	0.26	825
Series D	0.38	0.098	135	62.3	0.20	553
Series E	0.42	0.115	143	72.7	0.22	573
Series F	0.53	0.148	204	93.2	0.29	783
Series G	0.54	0.146	206	96.8	0.28	713
Series H	0.43	0.141	219	92.2	0.25	500
Series J	0.51	0.139	136	68.8	0.22	715
Series K	0.57	0.156	251	101.8	0.32	746
Average	0.49	0.138	187	85.1	0.26	682
Mean Variation from the Average, per cent	10.0	10.8	18.0	13.9	11.2	17.3

It will be noted that the endurance limit is about 50 per cent of the tensile strength, and about 25 per cent of the modulus of rupture obtained from the transverse test.

An attempt was made to correlate endurance limit with chemical composition as given in Table II. No consistent relation was found between endurance limit and combined carbon, graphitic carbon, ratio of combined carbon to graphitic carbon, total pearlite, and ratio of pearlite to graphitic carbon. A possible explanation of this lack of correlation may be the number of chemical and physical variables involved, internal stresses, or perhaps the previous history of the material from which the test bars were cast.

Correlation Between Tests not Including Endurance Limit:

Figure 12 shows the relation between Brinell hardness and tensile strength, and also modulus of rupture. There seems to be a fairly simple relation between these quantities and the tensile strength curve is fairly consistent. Figure 13 shows the relation between tensile strength, and also modulus of rupture, and also compressive strength. A few of the points do not lie very close to the curves.

Figure 14 shows the relation between energy of rupture and Brinell hardness, and also Rockwell hardness. Series *H* which contained 1.15 per cent of nickel seems to show particularly high toughness for a given hardness, as contrasted with series *K* which did not contain the nickel but was otherwise practically the same.

Figure 15 shows the relation between energy of rupture and modulus of rupture, and also tensile strength. A few of the plotted points do not lie very close to the curves.

FURTHER WORK ON CAST IRON

It is hoped that this paper will be looked upon merely as a preliminary survey and that further work on cast iron will be carried on in the near future.

It is quite possible that the maximum temperature reached by the cast iron in the melting process has an important influence on various properties. Perhaps in tests along this line the electric furnace should be used to melt the iron. It would be interesting to note the effect on endurance limit of superheating ordinary cupola iron in the electric furnace.

One of the matters which needs investigation is the influence of heat treatment of castings on the various properties. Perhaps some of the erratic results shown on the curves are due to the presence of internal stresses. A low-temperature anneal might be very effective in relieving these stresses.

Cast iron is very complex, both chemically and physically, and at the present time the information available is not precise enough to enable one to say just what the effect will be of varying certain chemical and physical factors. The various chemical ingredients, for instance, can be combined in so many different proportions that the problem is quite complicated.

Considerable work has been done in studying the effect of such alloys as nickel and chromium in producing certain desirable characteristics in cast iron. Such work is certainly of the greatest practical importance to the cast-iron industry.

Another investigation which might prove to be interesting would be a study of the effect of different pouring temperatures, and the effect of various cooling conditions in the molds.

DISCUSSION

Mr. Gillett.

MR. H. W. GILLETT[1] (*presented in written form*).—Mr. Kommers comments, on page 192 and at the top of page 193, on the many low points and the wide scatter. As he states, this is to be expected in the case of a material that is not homogeneous but consists of metal plus notches filled with graphite. The notches are just as characteristic of the material as is the metallic matrix. It seems to me that in plotting endurance data for an inhomogeneous material it is not safe to give so little weight to the low points.

Despite the sharp knees drawn in some of the curves, the general form of curves representing the data appears to be that of series *F* and *E* in Fig. 7. Curves of this type can be drawn through the uppermost points for each lot. If a parallel curve be drawn, displaced downward from such an upper curve by the width of the scatter, the enclosed band would, to my mind, more truly represent the data.

In Fig. 7, for example, four specimens of series *G* at close to or below the plotted endurance limit broke at 200,000 cycles or less. If those particular four specimens had been stressed at 15,000 lb. per sq. in. instead of 18,000 to 19,000 lb. per sq. in. is it not very likely that some of them would have failed short of 20 million cycles?

If a band were drawn to enclose all the plotted points for each series, it would indicate that series *F* would be safe at about 14,000 lb. per sq. in., series *G* at about 13,000 lb. per sq. in. and series *E* at about 12,000 lb. per sq. in. Would such an indication not be nearer the truth than the present plotting?

Casual examination of the plots would indicate that on such a basis of plotting the *safe* limits would be roughly as follows:

Series *A*........	15 000 lb. per sq. in.	Series *F*.......	14 000 lb. per sq. in.
Series *B*........	17 000 lb. per sq. in.	Series *G*.......	13 000 lb. per sq. in.
Series *C*........	14 000 lb. per sq. in.	Series *H*.......	18 000 lb. per sq. in.
Series *D*........	10 000 lb. per sq. in.	Series *J*.......	9 000 lb. per sq. in.
Series *E*........	12 000 lb. per sq. in.	Series *K*.......	20 000 lb. per sq. in.

If this view be taken, the conclusion that an estimation of probable endurance from the Brinell hardness should always be checked by direct experiment is strengthened. One might add that enough speci-

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[1] Chief, Division of Metallurgy, U. S. Bureau of Standards, Washington, D. C.

Mr. Gillett.

mens should be tested to show the scatter and thus differentiate between a material like series *E* and one like series *J*. As a material of construction, series *E* with its small scatter seems much more dependable in comparison with series *J* with its wide scatter than the endurance limits given in the paper would indicate.

These comments are offered for their bearing on the problem of plotting data such as these, not as adverse criticism on a very fine paper which gives much needed data.

Mr. MacPherran.

MR. R. S. MACPHERRAN[1] (*presented in written form*).—The paper just read by Mr. Kommers is very interesting and instructive but, as the author states, it should be regarded as merely beginning a subject and not ending it.

Variations in melting temperature and possibly variations in pouring temperature may be found to be as important as variations in composition. The effect of alloys opens another wide field of investigation, and it is not altogether improbable that heat treatment may develop along the lines already followed by steel products.

An interesting thing to note is that the irons represented in this series are with one exception not laboratory experiments or even test mixtures, but the regular product of large foundries operating under normal conditions. The extreme variation in endurance limits of from 11,800 to 24,100 lb. per sq. in. would indicate that cast iron is capable of considerable development and can no longer be classed under one heading. In reference to individual bars varying considerably in their fatigue value, I would point out that these specimens are of small diameter and consequently, if a flake of graphite happened to occur on the surface at the point of highest stress, it might affect the result to a greater degree than a test in which a bar of larger diameter were used.

I feel that this paper has opened up a great subject and I sincerely hope the work may be continued.

Mr. Coyle.

MR. F. B. COYLE.[2]—When I first heard of the program of fatigue testing which Mr. Kommers describes in his paper, I became immediately interested because of the fact that in our research laboratory we have three fatigue testing machines. Consequently, a program of fatigue testing of high-strength cast iron was initiated. Only one series has been completed to date. This material was similar to series *H* in Mr. Kommer's tests except that silicon was slightly lower (2.17 per cent) and nickel slightly higher (2.42 per cent). The tensile strength was 56,000 lb. per sq. in. and the fatigue limit 28,800 lb. per

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[1] Chief Chemist, Allis-Chalmers Manufacturing Co., West Allis, Wis.

[2] Research Metallurgist, International Nickel Co., Bayonne, N. J.

Mr. Coyle. sq. in. The type of specimen was the same as that used by Moore and Lyon, described last year.[1] The deviation of the various points from the curve in this series and those obtained in other series to date is much less than that shown in Mr. Kommers' tests. The points approach the curve very closely in the case of steel. When it is considered that cast iron is a material with an extremely low elastic limit, Mr. Kommers has demonstrated that such material can be stressed continuously well beyond the elastic limit without failure. In view of this fact it is apparent that Mr. Kommers has demonstrated the existence of a field worthy of extended investigation.

A study of Tables II and III shows that series *F* possesses only slightly higher tensile strength, compressive strength, Brinell hardness and fatigue limit than series *G*. Series *G* is slightly lower in both graphitic carbon (0.17 per cent) and silicon (0.10 per cent) than series *F*. While this slight difference in composition would account for some difference in physical properties, material corresponding to the composition of series *F* normally shows much higher physical properties, the tensile strength in particular usually being about 40,000 lb. per sq. in. and the transverse strength about 4500 lb. per sq. in.

Mr. McAdam. MR. D. J. MCADAM, JR.[2]—In regard to the curves in Figs. 6 and 7 of the paper, I agree with Mr. Gillett that it would be better to consider these as a band of scattered points. If a single line be used to represent the graph, it should be drawn as near as possible to the mean position, rather than through the lower edge of the scattered points. Each curve would thus indicate an average endurance limit, rather than the lowest endurance limit, as obtained by any individual specimen. Average endurance limits could then be compared with averages of other physical properties.

In some of the curves of Fig. 8, there are indications of a tendency toward reversal of curvature in the upper portion. Such reversal might be expected with this material, as its heat-conductivity is not great, and the speed of revolution is fairly high.

These suggested changes would make only slight changes in some of the endurance limits. Nevertheless, if line graphs be drawn, they should be drawn so as to represent as nearly as possible the true stress-cycle relationship.

Mr. MacKenzie. MR. J. T. MACKENZIE[3] (*by letter*).—I should like to draw attention to several relationships which were not mentioned in the paper.

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[1] H. F. Moore and S. W. Lyon, "Tests of the Endurance of Gray Cast Iron Under Repeated Stress," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 87 (1927).

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[3] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.

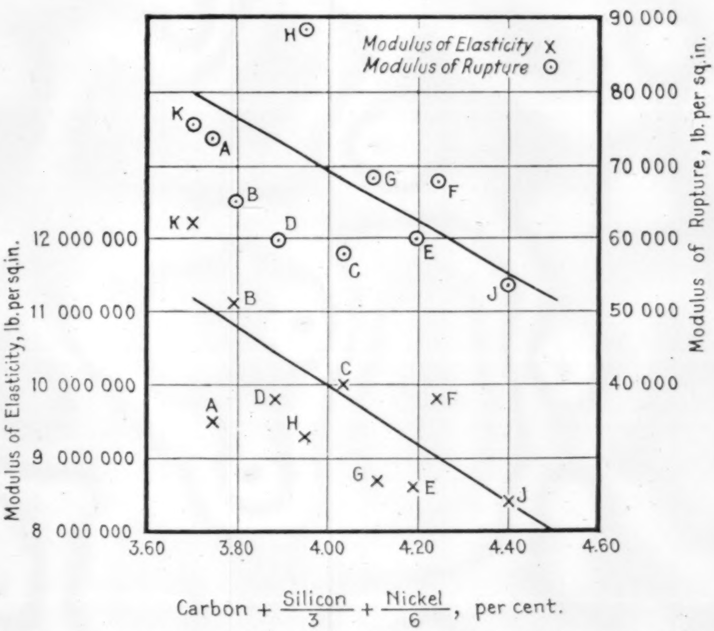

FIG. 1.—Curves Showing Relations Between the Modulus of Rupture and Modulus of Elasticity and the Chemical Composition of Cast Iron

The designations used on the curves are the same as those used on the figures in the paper.

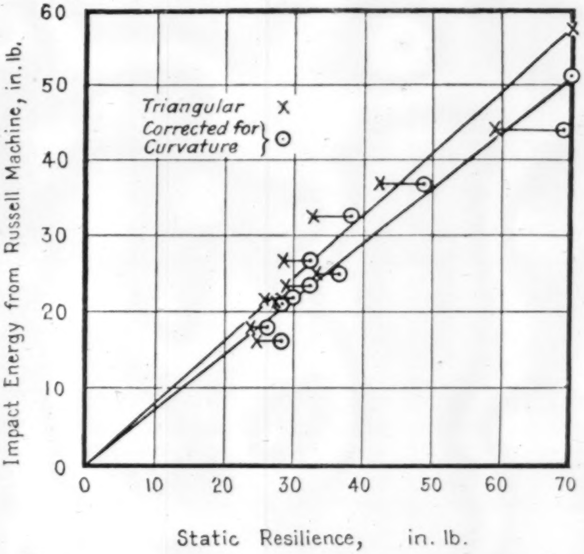

FIG. 2.—Curves Showing Relation Between Resilience Determined from Impact Energy and Static Resilience.

Mr. MacKenzie. In the accompanying Fig. 1 are shown the relations between the modulus of rupture and modulus of elasticity to the carbon, plus one-third of the silicon, plus one-sixth of the nickel. These show fairly good concordance with certain notable exceptions, especially series *A* in its exceedingly low modulus of elasticity, and series *H*, the high-nickel iron, for its very high modulus of rupture. Series *F*, which is the nickel-chromium iron, is stiff and strong for its other analysis. This is as it should be. The line drawn in roughly indicates the average.

The accompanying Fig. 2 shows a very interesting correlation between the resilience as determined from the impact energy in the Russell machine, and the static resilience calculated by means of the area under the bending curve. This may be expressed as one half the load multiplied by the deflection and divided by the volume of the test bar, or in another way it may be expressed as the square of the modulus of rupture divided by the modulus of elasticity and divided by the volume of the test piece.

I am indebted to Mr. Kommers for the planimetric determination of the excess of the full resilience over the triangular resilience, which is indicated in the figure. The average relation is shown by the lines drawn from the zero point for the two cases.

When we consider the large variables found by Mr. Kommers in the impact resistance of the several irons, it is not surprising that these ratios are not all on the line. The variation, however, varies from 60 per cent to 84 per cent, with an average of 73 per cent of the static resilience for the impact energy. This would seem to be a very useful figure in ordinary testing for the small foundry, and most of the large foundries do not have a machine adequate for breaking large specimens in impact.

Mr. Kommers. MR. J. B. KOMMERS.[1]—In regard to the drawing of a band on the *S-N* diagram as representing the results, I think the point is well taken. In the book which Mr. H. F. Moore and I published on "The Fatigue of Metals" we showed a curve with a shaded band for a material whose points showed considerable scatter. With a material as non-homogeneous as cast iron this procedure would be reasonable.

I have come into agreement with Mr. McAdam in one respect. In the *S-N* diagrams of the paper you will notice that the ordinates are drawn to a natural instead of a logarithmic scale. To the natural scale a line may be used to represent any number that is convenient, but to the logarithmic scale it can represent only a certain number or that number multiplied or divided by multiples of ten.

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[1] Professor of Mechanics, University of Wisconsin, Madison, Wis.

Mr. Kommers.

With regard to Mr. Coyle's results, apparently the ratio of his endurance limit of 28,800 lb. per sq. in. to a tensile strength of about 56,000 lb. per sq. in. is somewhere near the ratios reported in the paper.

For the *S-N* diagrams, I attempted to draw an average curve for all cases, but these did not always look alike. The upper curve in Fig. 7 indicates apparently that if a material is homogeneous the *S-N* diagram is curved from start to finish instead of having a sharp break. Of course I feel that the horizontal portion of the curve is the important part, and whether that is arrived at with two straight lines or with a continuous curved line is a matter of indifference.

(*Author's closure by letter*).—Since this paper was submitted one other experiment was tried. Some of the series *F* bars which contained 0.66 per cent of nickel and 0.25 per cent of chromium were heated to a temperature of 950° F. and held for three hours. All the tests except the impact and transverse tests were then applied as before. The following table gives the results:

Series	Tensile Strength, Small Specimen, lb. per sq. in.	Compression Test		Hardness Test			Fatigue
		Unit Stress at First Bend, lb. per sq. in.	Unit Stress at Ultimate, lb. per sq. in.	Brinell Number	Rockwell Number, "B" Scale		Endurance Limit, lb. per sq. in.
					At Center	At Mid-radius	
Series F as Cast	38 600	82 700	133 200	209.5	95.5	97.5	......
	35 400	82 800	129 100	214.5	97.5	97.5	......
	37 200	80 900	133 700	209.5	95.0	97.5	......
	......	......	......	207.0	97.0	98.0	......
	......	......	......	207.0	94.0	97.5	......
	......	......	......	......	....	....	......
Average	37 100	82 100	132 000	209.5	95.8	97.6	19 500
Series F Heated to 950° F. for 3 hours	35 400	74 700	136 500	212.0	94.0	97.0	......
	36 000	86 000	138 900	213.2	93.0	97.0	......
	36 500	84 900	131 000	212.0	95.0	97.5	......
	......	......	......	214.5	95.0	99.0	......
	......	......	......	217.0	96.0	97.0	......
	......	......	......	......	....	....	......
Average	36 000	81 900	135 500	213.7	94.6	97.5	19 700

It will be noted that for the heat-treated bars the results are almost exactly the same as those obtained on the same bars not heat treated, the endurance limit being actually increased slightly.

There is one matter mentioned in the paper which I think should receive further comment. Reference is made to a result reported in *Engineering*, London, in 1926 by C. H. Bulleid. He reports an endurance limit in reversed bending of 21,700 lb. per sq. in. on a cast iron the tensile strength of which was 25,500 lb. per sq. in. and the modulus of rupture 50,100 lb. per sq. in. In other words, the endurance limit was 85 per cent of the tensile strength and 43 per

Mr. Kommers. cent of the modulus of rupture. In my own tests the endurance limit showed an average result of 49 per cent of the tensile strength and 26 per cent of the modulus of rupture.

I was naturally skeptical about the correctness of Bulleid's result and wrote to him asking for more information. I had a reply from him, and he still believes that his results are correct, but that the tensile strength given may be low. This explanation is not satisfactory since his ratio of endurance limit to modulus of rupture was also very much too high. In fact, so far as I am aware, no investigator has reported even for steel a case in which the endurance limit in reversed bending was 85 per cent of the tensile strength.

I may say further that Bulleid and Almond published results on two more cast irons in 1927. These had endurance limits which were 54 and 34 per cent, respectively, of the tensile strength, and were therefore reasonable values.

I have discussed this matter at some length because the reported value of endurance limit happened to be the highest which had been reported for cast iron up to the time the present experiments were made.

It may be of interest to point out that Mr. MacKenzie found an average carbon for series *A* of 3.37 per cent instead of 3.30 as given in the paper. Series *A* also contained less than 0.01 per cent of nickel. He also found that series *G* had 0.10 per cent of chromium, and 0.09 per cent of nickel.

At Mr. MacKenzie's suggestion, I am giving below the values of the deflections in the transverse test after being reduced to a diameter of specimen of 1.25 in.:

	Maximum Deflection, in.		Maximum Deflection, in.		Maximum Deflection, in.		Maximum Deflection, in.		Maximum Deflection, in.
Series A	0.161 0.128 0.161 0.159 0.143	Series C	0.121 0.095 0.110 0.112 0.111	Series E	0.121 0.124 0.134 0.140 0.135	Series G	0.132 0.161 0.108 0.160 0.160	Series J	0.116 0.144 0.129 0.102 0.136
Average	0.150	Average	0.110	Average	0.131	Average	0.144	Average	0.125
Series B	0.114 0.109 0.108 0.108 0.111	Series D	0.125 0.100 0.118 0.111 0.121	Series F	0.148 0.126 0.140 0.115 0.109	Series H	0.175 0.156 0.180 0.185 0.184	Series K	0.126 0.115 0.116 0.096 0.115
Average	0.110	Average	0.115	Average	0.128	Average	0.176	Average	0.114

THE EFFECT OF SULFUR ON THE PHYSICAL PROPERTIES OF GRAY IRON

By Edwin K. Smith[1] and Fred. B. Riggan[2]

Synopsis

This paper deals with a series of experiments, designed to show the practical effect of various percentages of sulfur in ordinary gray iron.

About two hundred trial charges were run, to get sufficient bars containing increasing amounts of sulfur, with all other elements substantially uniform.

Curves show that, contrary to general opinion, strength and fluidity of gray iron are not affected by sulfur, even in great excess. Brinell hardness and combined carbon are increased by sulfur, while machineability is correspondingly decreased.

Microscopic examination shows increasing sulfide areas, believed by the authors to be sulfide of iron in solution in sulfide of manganese, with gradual transition from normal gray iron to austenitic hard iron. There is evidence of solution of sulfide of iron in carbide of iron, and where sulfur is very high of precipitation of iron - iron sulfide eutectic.

There is strong evidence that sulfur can be materially reduced by the addition of manganese with the charge.

The authors believe that many defects attributed to sulfur have been due to other causes, and that with a proper percentage of manganese, sulfur has little effect on gray iron, at least up to 0.180 per cent.

For years many of the foundry ills, especially hard iron, blow holes, excessive shrinkage, and weak iron have been attributed to sulfur. A great deal of research has been carried out on the possible ways in which sulfur can combine with iron, but, until recently, little of the information secured has been in a form to make it available to the foundry. In general, the foundryman, as well as the buyer of gray iron, has demanded low sulfur, although in recent years the sulfur content in high-grade gray iron has been steadily increasing. At present there is a wide difference of opinion as to the practical effects of sulfur. The great majority of research articles indicate the harmlessness of sulfur *when manganese is in proper ratio*, while in most foundries, objection is raised to high sulfur. These differences of opinion are reflected in specifications. Some specify physical requirements only, some specify a sulfur content under 0.10 per cent, some

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[1] Metallurgist, Stockham Pipe and Fittings Co., Birmingham, Ala.

[2] Assistant Metallurgist, Stockham Pipe and Fittings Co., Birmingham, Ala.

a sulfur content under 0.12 per cent, while in a Diesel engine specification, the requirement appears, "sulfur as high as possible."

In view of this condition, it seemed worth while to run a series of trial heats, to determine the actual effect of sulfur in all proportions on ordinary commercial gray iron by cupola process.

EXISTING DATA ON SULFUR IN GRAY IRON

There is considerable difference of opinion as to the state of sulfur in gray iron.

Arnold and Waterhouse[1] decided that when iron sulfide is present, it exists as a separate constituent, not as a solid solution in manganese sulfide.

Levy in 1908 published results giving the conclusion that iron sulfide in eutectic and resultant sulfide films about particles of carbide prevent the decomposition of the latter and graphitization; therefore hard iron results.[2]

In 1910 Stead stated that more than 0.10 per cent of iron sulfide crystallizes in iron carbide as small inclusions, and can be brought out by heat tinting.[3]

Rohl determined the melting point of manganese sulfide as 1620° C. (2948° F.). He made up an iron sulfide - manganese sulfide diagram, showing formation of a compound $Fe_3Mn_2S_5$. He also proved the reaction Mn plus FeS equals MnS plus Fe to be reversible.[4]

McCance disagreed with this, but Herty and True confirmed the reversibility of this reaction.[5]

Rosenhain states that iron sulfide solidifies at 1188° C. (2170° F.) and is squeezed to the grain borders.[6]

On every point there seems to be lack of agreement. Investigators state that iron sulfide is—and is not—soluble in austenite, or in manganese sulfide. That sulfide of iron is easily identified as yellow or brown flakes, also that it is never found as such.

Hurst, in his recent book[7] sums up the opinions of the majority of recent workers, although his conclusions are not universally accepted: "sulfur in commercial cast iron, in the presence of sufficient manganese, exists almost wholly as sulfide of manganese. In this form it is practically without influence on the structure and properties of cast iron. In the absence of sufficient manganese the sulfur exists as the iron sulfide, which is soluble in the liquid iron, and which has the tendency to prevent the formation of graphite."

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[1] Arnold and Waterhouse, *Transactions*, Iron and Steel Inst., Vol. 63, p. 126 (1903).

[2] Levy, *Metallurgie*, Vol. 5, p. 327 (1908).

[3] Stead, British Assn. Advancement of Science (1910).

[4] Rohl, *Carnegie Scholarship Memoirs* (Iron and Steel Inst.), Vol. 4, p. 28 (1912).

[5] Herty and True, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 71, p. 540 (1925).

[6] Walter Rosenhain, " 'Slag Enclosures' in Steel," *Proceedings*, Fifth Congress, Inter. Assn. Testing Mats. (1909).

[7] Hurst, "Metallurgy of Cast Iron," p. 107 (1926).

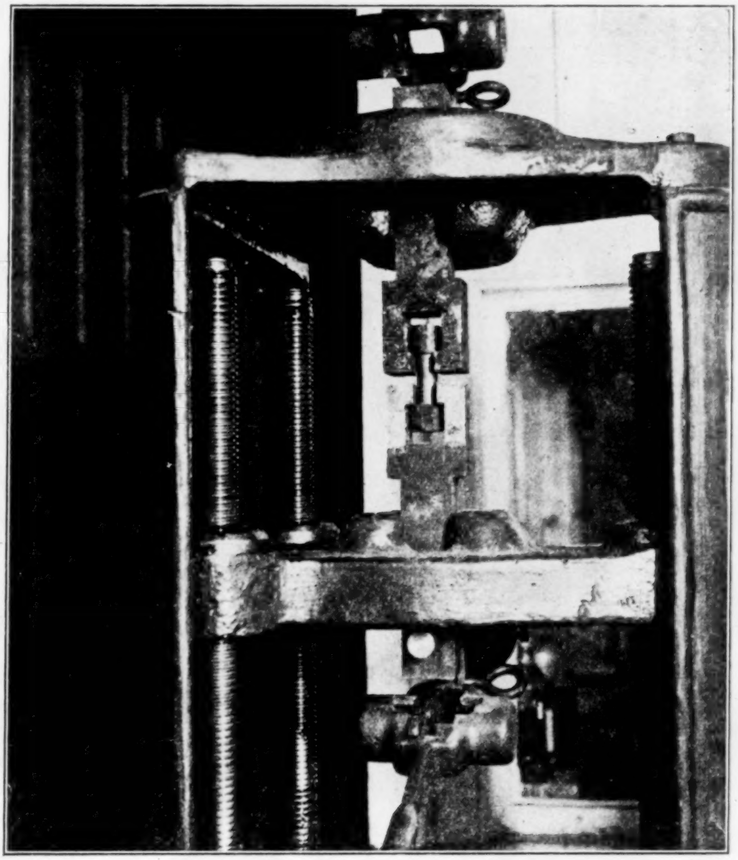

FIG. 1.—Detail of Apparatus for Tension Tests.

PROCEDURE

It would be impossible to try out the effect of sulfur in all different compositions. Therefore the composition was kept constant, with varying amounts of sulfur. The objective was to work with ordinary commercial metal and duplicate the usual temperature, height of bed, and time of cooling.

The raw materials were broken up into uniform sizes, and the coke was hand picked to insure absolute uniformity of melting con-

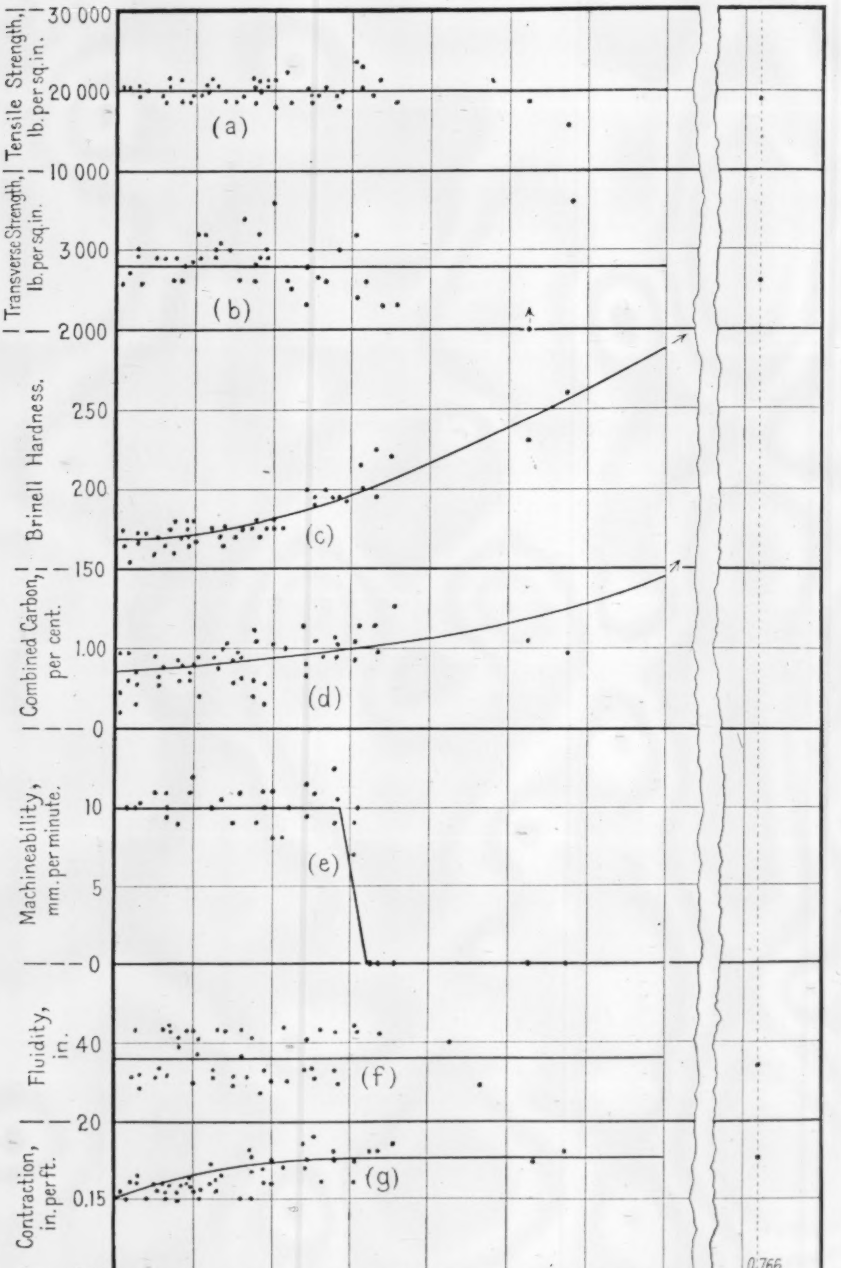

FIG. 2.—Showing Results of Tests on Effect of the Presence of Sulfur on Cast Iron.

ditions. Each charge was tapped into a hot ladle, from which were poured the arbitration, shrinkage, and fluidity bars. All samples were poured at the same temperature (1460° C. (2660° F.), as determined by a Leeds & Northrup optical pyrometer).

Analyses of the mixtures showed the following:

	SILICON, PER CENT	MANGANESE, PER CENT	SULFUR, PER CENT	PHOSPHORUS PER CENT
Charged...........	2.20	0.50	0.050 - 2.0	0.55
Poured............	1.75 - 2.00	0.32 - 0.36	0.054 - 0.766	0.55 - 0.62

Total carbon in the bars ran 3.45 to 3.60 per cent.

The sulfur additions were made with ferrous sulfide sticks, and the manganese was added in the form of 80-per-cent ferro-manganese.

TEST RESULTS

Figure 1 shows the apparatus used with special jaws for testing the gray iron specimens, so as to eliminate side strains. The jaws are pivoted, and are free to move, as is the test bar.

The graphs of Fig. 2 give the results of all experiments. Any bars which did not conform to the standard analysis were discarded.

Curves (*a*) and (*b*) show that sulfur has no effect on either transverse or tensile strengths. This is in agreement with results reported by Hamasumi.[1]

Curves (*c*) and (*d*) show that Brinell hardness and combined carbon rise gradually with increasing sulfur. This confirms Schuz statement that combined carbon and Brinell hardness follow a straight line law.[2]

Curve (*e*) shows that with increasing sulfur content the machineability remains the same until a sulfur content of about 0.200 per cent is reached, above which point, with the low manganese used in this series of experiments, the specimen is commercially unmachineable.

Curve (*f*) shows that it is impossible to detect any significant change in fluidity with sulfurs varying from 0.054 to 0.766 per cent, the pouring temperature remaining the same at 1460° C. (2660° F.).

Curve (*g*) shows that the contraction curve rises slightly up to a point about 0.200 per cent sulfur, and then remains constant.

A further series of experiments on shrinkage showed that moderately high sulfur had little effect, but purposely oxidized iron gave very great shrinkage, thus agreeing with Mr. Moldenke that oxidation causes trouble often attributed to sulfur.

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[1] Hurst, "Metallurgy of Cast Iron," p. 239 (1926).

[2] Schuz, *Stahl und Eisen*, Vol. XLIII, May, 1922.

DISCUSSION

Desulfurization by Means of Manganese:

The effect of sulfur on gray iron depends so greatly on the amount of manganese in the melt that the effect of one can hardly be considered without the other. In these experiments, it seemed best to hold constant the manganese as well as the other elements, except sulfur, in order that fluctuations in manganese might not mask the effects of the sulfur.

Shaw states,[1] "we have never found much loss of sulfur under ordinary conditions by raising the manganese." In proof of this he shows that certain bars gave no reduction of sulfur on standing for some time. The reason for this certainly seems to be due to the fact that the two lots of iron did not contain enough manganese to combine with carbon and sulfur. Furthermore, Mr. Shaw quotes Meierling and Dennecke[2] who find considerable desulfurization due to segregation of mixed crystals rich in iron and manganese sulfide.

Desulfurization by manganese is proved by Herty and True[3] who find the greater the excess of manganese, the lower the sulfur in the metal.

Herty and Gaines[4] have worked out conditions under which manganese will desulfurize iron, arriving at the interesting conclusion that desulfurization takes place when the product of the percentage of manganese times the percentage of sulfur is above 0.070.

Our experiments gave an unlooked for confirmation of desulfurization by manganese, as we had considerable difficulty in obtaining a melt with high manganese and high sulfur. Almost invariably, the manganese would combine with the sulfur and slag off. If a large excess of sulfur was charged, the manganese would be reduced to a very low figure, and hard iron resulted. If a large amount of sulfur with correspondingly high manganese was charged, the resulting iron had low sulfur and low manganese. From the authorities quoted and from our experiments we feel safe in assuming that in ordinary cupola practice, a considerable proportion of the sulfur can be combined with manganese to form manganese sulfide, which partly goes into slag and partly remains in iron as comparatively harmless manganese sulfide globules.

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[1] Shaw, *Transactions*, Am. Foundrymen's Assn., Vol. 34, p. 810 (1926).

[2] Meierling and Dennecke, *Giesserei Zeitung*, April 1, 1926.

[3] Herty and True, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 71, p. 540 (1925).

[4] Herty and Gaines, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 75, p. 434 (1927).

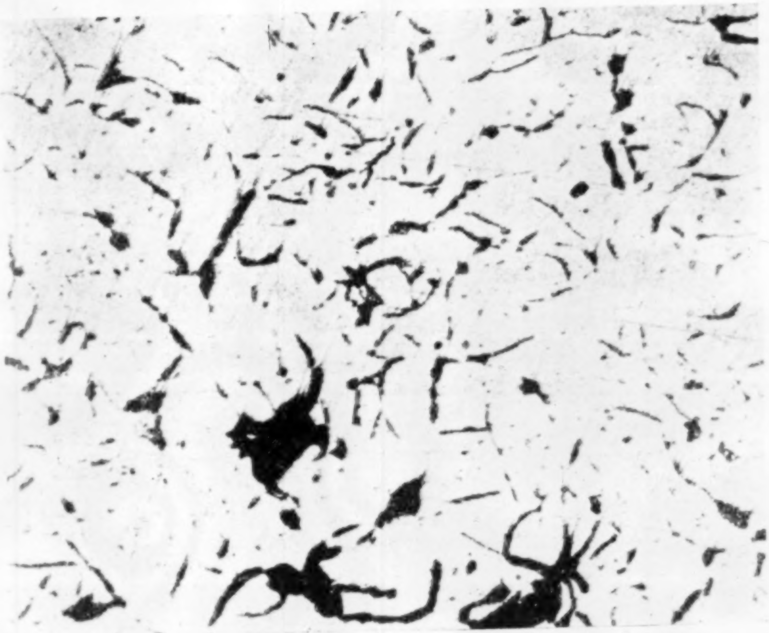

FIG. 3.—Photomicrograph of Gray Cast Iron. Sulfur 0.059 per cent. Unetched. Shows normal gray iron (× 100).

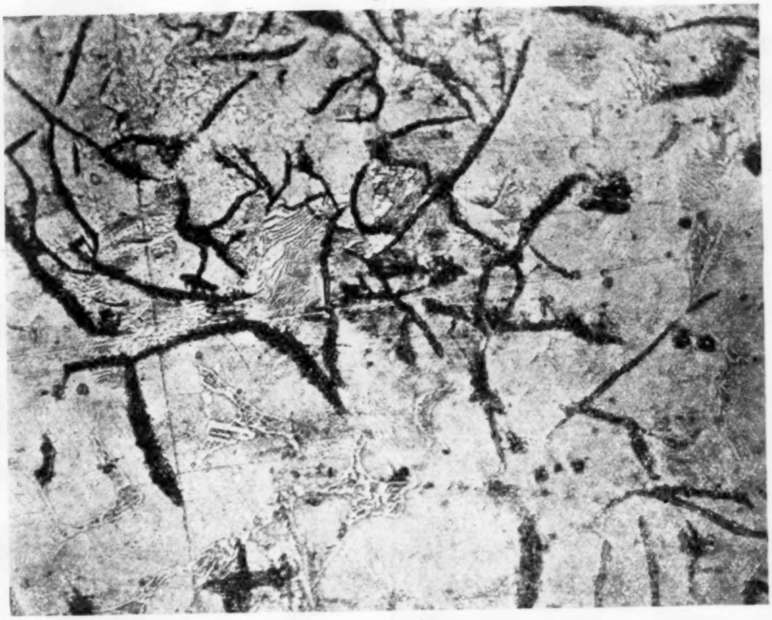

FIG. 4.—Photomicrograph of Gray Cast Iron; same as Fig. 3. Lightly etched with picric acid in alcohol (× 300).

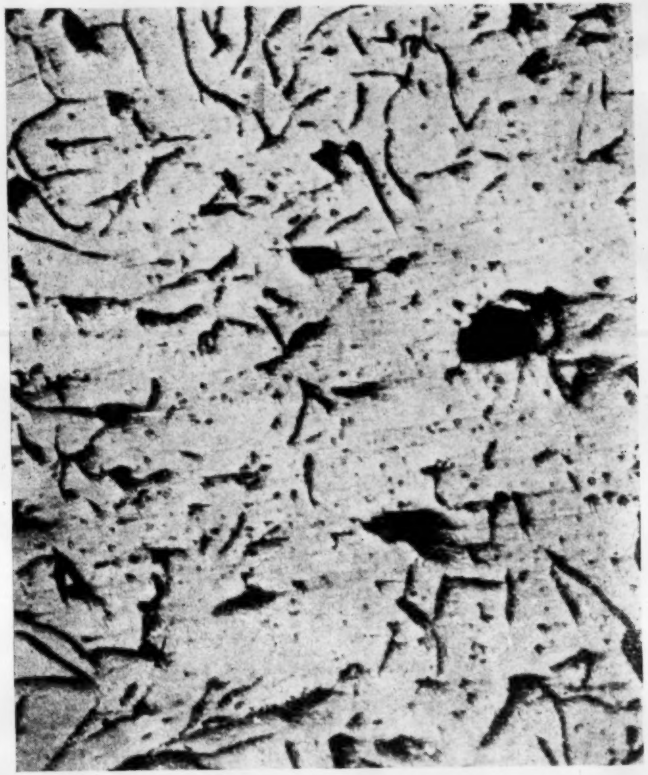

FIG. 5.—Photomicrograph of Gray Cast Iron. Sulfur 0.145 per cent. Unetched. Shows normal gray iron. Small globules manganese sulfide (× 100).

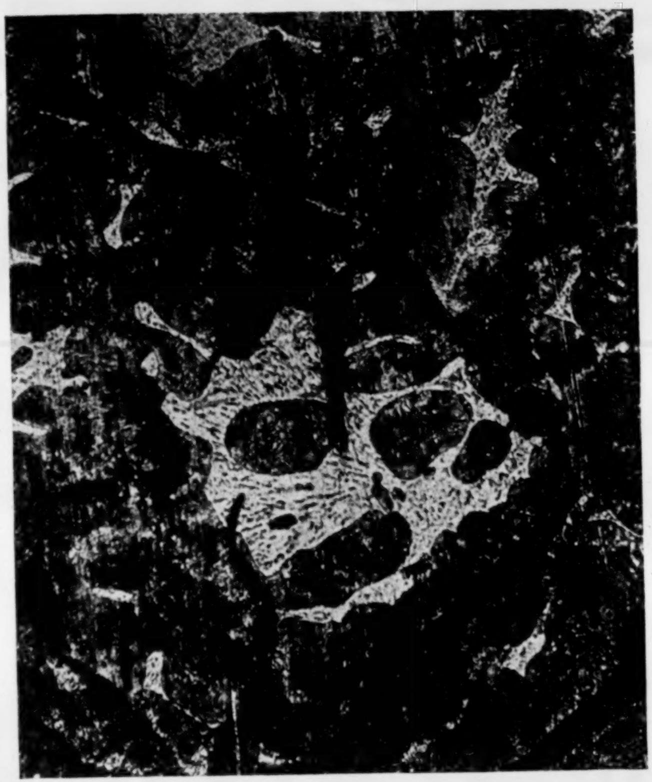

FIG. 6.—Photomicrograph of Gray Cast Iron; same as Fig 5. Deeply etched with picric acid in alcohol (× 300).

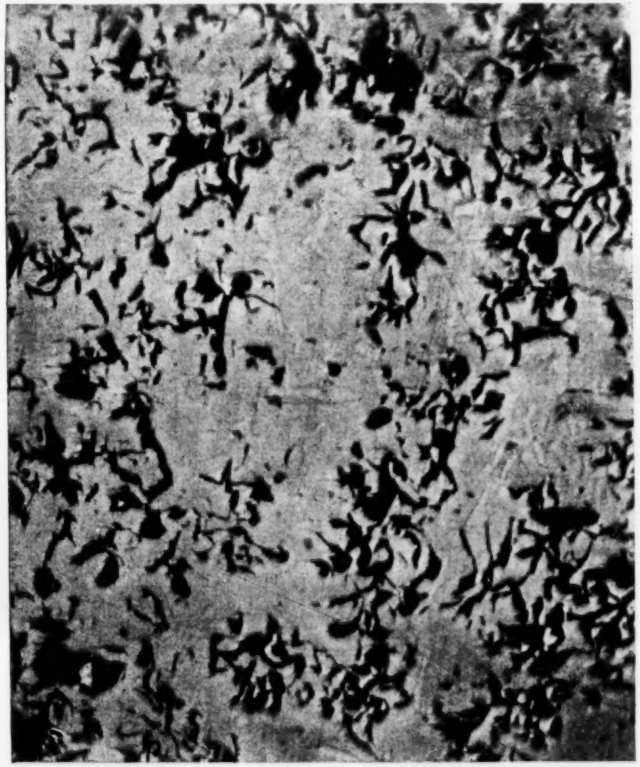

FIG. 7.—Photomicrograph of Gray Cast Iron. Sulfur 0.337 per cent. Unetched. Shows much finer graphite, due to increasing sulfur (× 100).

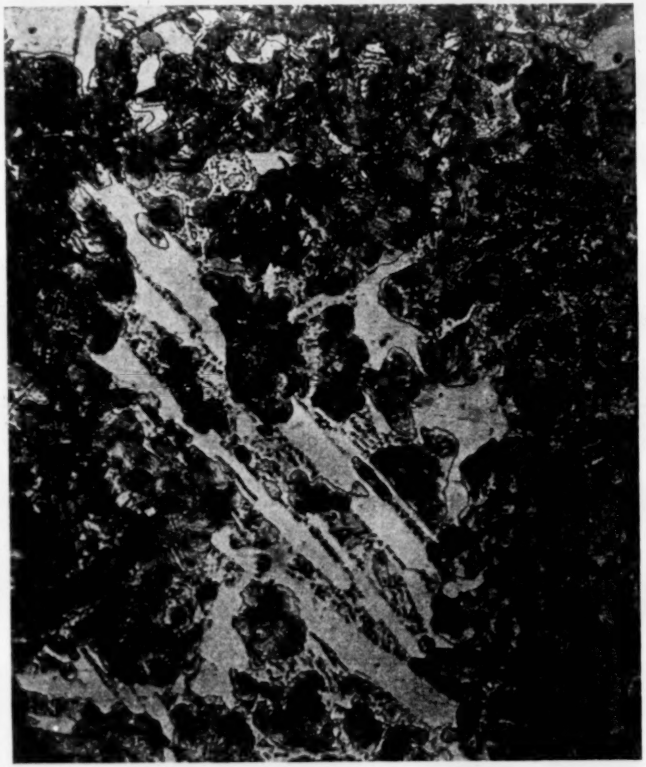

FIG. 8.—Photomicrograph of Gray Cast Iron; same as Fig. 7. Etched with picric acid in alcohol. Shows considerable combined carbon, and inclusions of manganese sulfide. This bar could not be drilled (× 300).

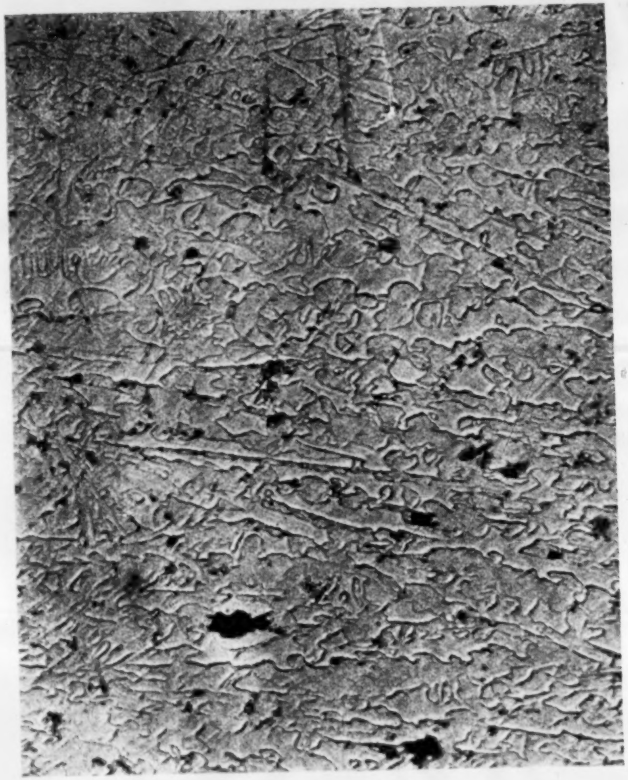

FIG. 9.—Photomicrograph of Gray Cast Iron. Sulfur 0.766 per cent. Unetched. Shows dendritic structure, and small nodules of carbon (× 100).

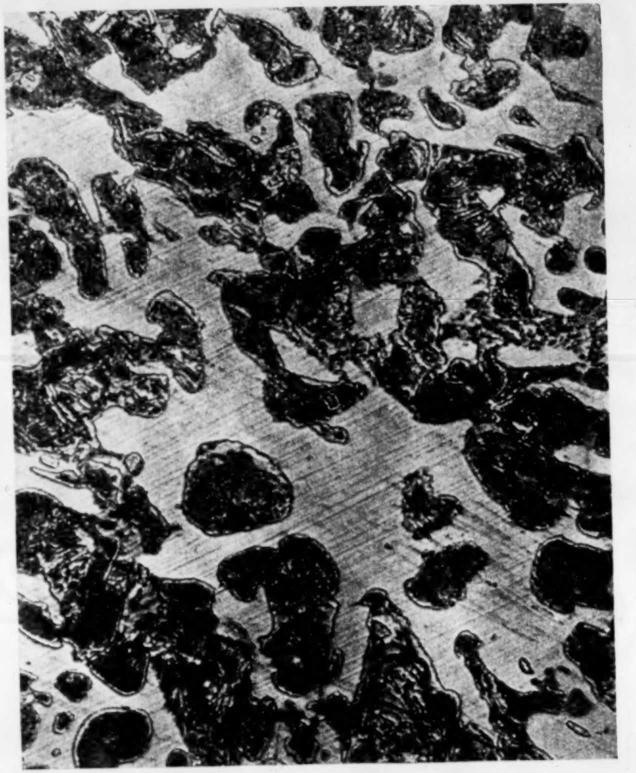

FIG. 10.—Photomicrograph of Gray Cast Iron; same as Fig. 9. Etched with picric acid in alcohol. Shows moderate number nodules manganese sulfide. Grains of pearlite surrounded by hard sheath, which we consider iron - iron sulfide eutectic (× 300).

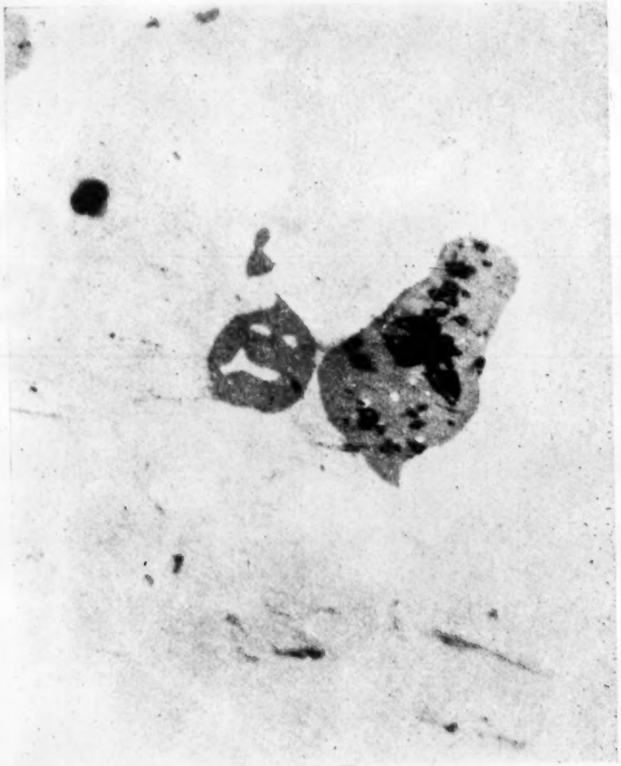

FIG. 11.—Photomicrograph of Gray Cast Iron. Sulfur 1.21 per cent. Unetched. Shows globules of manganese sulfide containing a second phase, which may be iron - iron sulfide eutectic and iron sulfide (× 300).

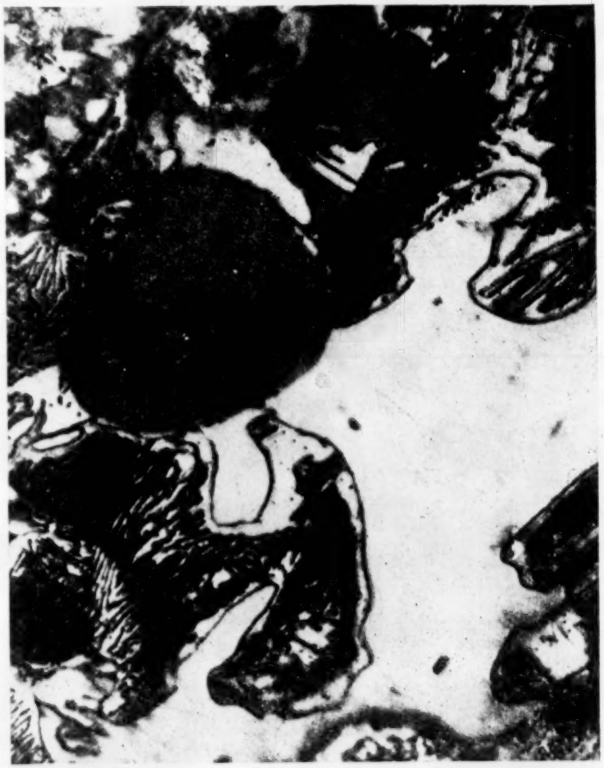

FIG. 12.—Photomicrograph of Gray Cast Iron; same as Fig. 11. Etched with picric acid in alcohol. Shows grains of pearlite completely surrounded by dotted white area, which we consider iron - iron sulfide eutectic. Large nodules manganese sulfide - iron sulfide (× 500).

Distribution of Sulfur:

Our opinion is, that in the molten iron, when sufficient manganese is present, practically all of the sulfur combines with the manganese, to form manganese sulfide, some of which remains in the solid metal as inclusions. The remainder rises to the top as slag and is eliminated from the metal, the completeness of this desulfurization depending largely on the excess of manganese over sulfur, the temperature of the metal, and the time available for the reaction.

When there is not sufficient manganese to combine with the sulfur, most of the surplus sulfur remains in solid solution in iron and iron carbide.

When sulfur is in great excess over manganese, the iron carbide becomes supersaturated, and more or less heavy films of iron - iron sulfide eutectic separate around the grains of pearlite, preventing graphitization, as suggested by Levy, Osmond, and others.

Some excess iron sulfide undoubtedly dissolves in globules of manganese sulfide, forming two separate phases, as noted by Rawdon.[1]

Microstructure:

A great deal can be learned of the various forms of sulfur in gray iron through studying its microstructure. In Figs. 3 to 12 are given micrographs of the iron with increasing sulfur. A similar set of sulfur prints gave progressively darker prints, with no evidence of marked segregation.

CONCLUSIONS

The following conclusions in respect to sulfur in gray iron are drawn. It should be borne in mind that these experiments were purposely run with the percentage of manganese fixed at rather a low figure:

1. Sulfur in gray iron seems to be distributed as follows: Any manganese left over after combining with the carbon combines with sulfur to form manganese sulfide, which rises to surface, if the temperature and time permit. The remainder of the sulfur is in solid solution in carbide and manganese sulfide nodules. If the sulfur is very high, the excess is precipitated from carbide as iron - iron sulfide eutectic, and from manganese sulfide as some form of iron sulfide.

2. Sulfur has practically no effect on tensile and transverse strengths, and on fluidity.

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[1] Herty and True, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 71, p. 547 (1925).

3. Sulfur increases Brinell hardness, and combined carbon, the increase not being marked until sulfur reaches 0.180 per cent.

4. Unless there is sufficient manganese present, sulfur up to 0.190 per cent does not affect machineability, but beyond that figure, the metal becomes unmachineable.

5. Contraction is increased gradually up to 0.190 per cent of sulfur and from there remains constant.

6. A considerable amount of sulfur can be removed from gray iron by using manganese in proper proportions in the charge, provided time and temperature are sufficient to effect separation.

DISCUSSION

Mr. Lowry.

Mr. E. J. Lowry.[1]—Can the authors explain how they arrived at the conclusion that there is a desulfurization, given sufficient time and temperature and sufficient manganese present?

Mr. Riggan.

Mr. F. B. Riggan.[2]—We had considerable difficulty in obtaining bars of both high manganese and high sulfur. We tried several times to get a bar of 1.00 per cent sulfur with a correspondingly high manganese, to show that it would be machineable, but in our experiments we were unable to get that combination of both high manganese and high sulfur.

Mr. Lowry.

Mr. Lowry.—But how do you arrive at the conclusion that there is desulfurization when sufficient manganese is present?

Mr. MacKenzie.

Mr. J. T. MacKenzie.[3]—I think I can answer Mr. Lowry's question. Some time ago I received a trial lot of pig iron with 0.700 per cent of sulfur in it. The carbon and silicon were low, and the manganese was still lower. Running it through the cupola I was a little surprised to find that the sulfur came out at 0.400 per cent instead of 0.700 per cent, or even higher, as we would expect the normal sulfur pick up. I melted some more of that iron, using pitch coke, in which I succeeded in driving another 1 or 1.5 per cent of carbon into it, putting it out at 3.5 instead of about 2.25 per cent of carbon, and adding some ferro-silicon and enough manganese to give it 3 per cent in the finished casting. I tapped the mixture at below 0.060 per cent sulfur with only 1.00 per cent of manganese, and the accumulation of manganese sulfides was so severe in the basin of the cupola that we had considerable difficulty in keeping the tap hole open. We would tap it out and the iron would immediately begin to run, but it would soon dwindle away and we had to rod the hole open; and when the ladle was filled there was fully 2 in. of very evil-looking slag on top of the metal. The metal underneath that poured perfectly freely and gave no trouble at all; in fact, it was a beautiful soft gray iron, but we were all rather surprised to find that the sulfur was only a tenth of that originally charged in the cupola. With the original experiment we lost only 0.300 per cent of sulfur because we did not have enough manganese to take care of the sulfur or enough

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[1] Consulting Metallurgist, Detroit, Mich.

[2] Assistant Metallurgist, Stockham Pipe and Fittings Co., Birmingham, Ala.

[3] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.

Mr. MacKenzie.

carbon for a low freezing point; consequently the fluidity was insufficient to allow the manganese sulfide to escape.

Mr. Gillett.

MR. H. W. GILLETT.[1]—I should like to ask what were the methods of test for fluidity and machineability?

Mr. Riggan.

MR. RIGGAN.—The fluidity bars were wedge-shaped bars 80 in. long cast in open molds at temperatures constant at 2660°F. To determine machineability we made a drill test, using a $\frac{3}{8}$-in. high-speed drill with a 160-lb. weight and a drill speed of 475 r.p.m. At first we started out to drill through the test piece, but found that the drill would dull and we could not get very good results. We then measured the penetration in a given time and obtained satisfactory results.

Mr. Harrington.

MR. R. F. HARRINGTON.[2]—If I understand correctly, the increase in sulfur showed no effect on the tensile strength or transverse load; it did however show an effect on the combined carbon and on the Brinell hardness. I am interested to know whether or not microscopic examination showed a definite relation between the microstructure and the combined carbon.

Mr. Smith.

MR. E. K. SMITH[3] (*author's closure by letter*).—In answer to Mr. Lowry's question as to evidence of desulfurization by excess manganese, when time and temperature were sufficiently high: We found such desulfurization both in the cupola and in the ladle. As Mr. Riggan stated, a high manganese content in the charge invariably reduced the sulfur. For instance, if sulfur was charged at 0.800 per cent and manganese at 0.50 per cent most of the sulfur was taken up by the iron, which ran about 0.700 per cent sulfur. If, however, sulfur was charged at 0.800 per cent, and manganese at 2.00 per cent, we would get a low sulfur in the casting, which seems proof of desulfurization in the cupola. In the ladle, results were even more emphatic. To be specific, we give one case: Very hot, high-sulfur gray iron was caught in the ladle and a test bar poured, showing sulfur, 0.128 per cent; manganese, 0.50 per cent. An excess of 80-per-cent ferro-manganese was added, stirred, and permitted to stand still until necessary to pour. The sulfur was reduced to 0.053 per cent, with manganese 1.82 per cent. Lesser desulfurization can be obtained with less manganese. To anyone wishing to prove desulfurization by manganese, this experiment is easily made, and rather convincing.

Answering Mr. Harrington: Microstructure of samples showed increasing areas of carbide with increasing percentages of sulfur, and

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[1] Chief, Division of Metallurgy, U. S. Bureau of Standards, Washington, D. C.

[2] Metallurgist, Hunt-Spiller Manufacturing Corporation, South Boston, Mass.

[3] Metallurgist, Stockham Pipe and Fittings Co., Birmingham, Ala.

Mr. Smith. increasing combined carbon (by analysis). A bar with 0.145 per cent of sulfur contained 0.90 per cent of combined carbon, with no appreciable carbide visible in the microscope. A micrograph of a sample containing 0.170 per cent of sulfur with combined carbon 1.10 per cent (by analysis) showed traces of carbide needles. A micrograph of a sample with 0.337 per cent of sulfur (shown in paper) shows a large area of carbide, with actual analysis of 1.98 per cent combined carbon.

A STUDY OF CENTRIFUGALLY-CAST PIPE (METAL-MOLD PROCESS)

VERSUS

SAND-CAST PIPE

By F. N. Menefee[1] and A. E. White[2]

Synopsis

The object of this investigation was to determine the relative merits of sand-cast pipe and pipe centrifugally cast in metal molds. The scope of the investigation included physical, chemical and metallurgical tests on 48 pieces of pipe, 4 of which were selected by the Water Board and 44 picked at random by the authors at one of the yards of the Detroit Water Board.

No attempt has been made in this report to cover such phases as installation, maintenance, tests of joints and other matters of similar character.

A brief summary of the findings is as follows:

Importance of Tests	Tests	Favor
Primary	Tension	Centrifugal
Primary	Variation in Thickness	Centrifugal
Primary	Removal of Strains	No decision
Primary	Impact	Sand Cast
Primary	Radial Compression	Sand Cast
Secondary	Radial Deflection	Centrifugal
Secondary	Transverse	Sand Cast
Secondary	Inside Smoothness	Centrifugal
Secondary	Cutting	Centrifugal
Secondary	Metallographic	Centrifugal
Secondary	Hardness	Sand Cast
Secondary	Chemical	No decision

The twelve headings shown above represent the main subdivisions of the investigation. The first five are, in the judgment of the authors, the ones of outstanding importance.

Introduction

This investigation was undertaken at the request of the Water Board of the City of Detroit to determine the relative merits of sand-cast pipe and pipe centrifugally cast in metal molds. The work involved physical, chemical and metallographic tests on 48 pieces of pipe, 4 of which were sent to the laboratory by the Water Board and 44 of which were picked at random from the Detroit yards of the Water Board by the authors. The investigation involved the following determinations: tension, impact, radial compression, radial deflection, transverse and hardness tests; likewise, chemical and metallographic tests and also findings with respect to variation in thickness, the extent to which strains had been removed, inside smoothness and ease of cutting were given careful consideration. This paper does not discuss the resistance to corrosion of the pipe. This is a study in itself and the U. S. Bureau of Standards is engaged in an extensive investigation in this field.

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[1] Professor of Engineering Mechanics, University of Michigan, Ann Arbor, Mich.

[2] Professor of Metallurgical Engineering and Director of Department of Engineering Research, University of Michigan, Ann Arbor, Mich.

DESCRIPTION OF METHODS OF MANUFACTURE

The sand-cast process is the older method of manufacturing pipe. It has been in use for over 100 years. It consists in placing within iron flasks core boxes covered with hay, straw or shredded rope protected on the outside by sand and clay. These core boxes are baked before they are placed in the flasks in order to give their surface proper hardness. The iron is poured through suitable gates between the flask and the core, resulting in the formation of the pipe. After the metal has been poured into the mold, the flask is opened and the pipe is removed while quite hot.

In the metal-mold centrifugal process, "the pipe is cast in a revolving steel cylindrical mold the interior surface of which corresponds in every detail with the exterior of the pipe to be cast. Molten iron is poured through a spout extending through to the end of the mold and as the iron begins to flow the cylinder develops two motions. One is a revolving motion by which the iron is distributed centrifugally against the inner circumference. The other is a longitudinal movement by which the mold is drawn back, permitting the end of the spout to deliver molten metal along the inner surface of the revolving mold until the spout emerges from the end at which it entered. Inasmuch as the pipe is cast in a chill mold, it must be annealed before it is ready for service." Throughout the paper, all reference to "centrifugally-cast pipe" is to pipe made by the process just described.

TEST PROCEDURE AND TEST DATA

The study was made under twelve sub-divisions as follows:

Tests of Primary Importance:

Variations in Thickness
Tension Test
Impact Test
Radial Compression Test
Removal of Strains

Tests of Secondary Importance:

Radial Deflection Test
Inside Smoothness
Cutting Test
Metallographic Properties
Transverse Test
Hardness Test
Chemical Composition

These sub-divisions were chosen because they had either an influence on the service which the pipe would render or seemed to simulate the various service conditions which are imposed on the pipe.

While the report was in preparation, a paper dealing with practically the same subjects, by Arthur N. Talbot and Frank E. Richart, was read before the Society[1] at its annual meeting in June, 1926. Reference thereto will disclose that impact, resistance to internal pressure (tension) and flexure are given prominent consideration, the latter corresponding in a general way to the radial compression, radial deflection and transverse bending tests herein described.

The apparatus used was standard or such as is commonly known. The tension tests were run on a Riéhle 50,000-lb. machine sensitive to 6 lb. at zero load. The radial compression tests were run on a Riéhle 200,000-lb. machine. The impact tests were run on 100-ft-lb. and 25-in-lb. Olsen Izod type machines, and the transverse tests on an Olsen machine commonly used for cast-iron arbitration test bars.

Marking:

Forty-four pieces of pipe were selected at random from one of the Detroit yards; eleven 6-in. centrifugal, eleven 6-in. sand cast, eleven 8-in. centrifugal and eleven 8-in. sand cast. The data here reported do not include those on the 4 specimens tested previous to the tests on the 44 samples, since the procedure was changed slightly. Nothing in the tests on the 4 pieces ran contrary to the results on the 44. In each size were old and new pipe; old, meaning pipe that had been on hand for some time; new, meaning from recent shipments. They were numbered as follows:

6CO1	6SO1	8CO1	8SO1
6CO2	6SO2	8CO2	8SO2
6CO3	6SO3	8CO3	8SO3
6CO4	6SO4	8CO4	8SO4
6CO5	6SO5	8CO5	8SO5
6CN1	6SN1	8CN1	8SN1
6CN2	6SN2	8CN2	8SN2
6CN3	6SN3	8CN3	8SN3
6CN4	6SN4	8CN4	8SN4
6CN5	6SN5	8CN5	8SN5
6CN6	6SN6	8CN6	8SN6

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[1] Arthur N. Talbot and Frank E. Richart, "A Study of the Relation Between Properties of Cast-Iron Pipe Tested Under Impact, Internal Pressure, and Flexure, and the Corresponding Properties Found in Several Kinds of Test Specimens Taken Therefrom," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 185 (1926).

6 or 8 for size; *C* for centrifugal; *S* for sand cast; *O* for old; *N* for new; 1, 2, 3, etc., for the number of the specimen.

Variation in Thickness:

In the specifications of the American Water Works Association, adopted May 12, 1908, "Allowable Variation in Thickness" is discussed in Section 3 as follows:

> For pipe whose standard thickness is less than 1 in., the thickness of metal in the body of the pipe shall not be more than 0.08 in. less than the standard thickness, and for pipe whose standard thickness is 1 in. or more, the variation shall not exceed 0.10 in., except that, for spaces not exceeding 8 in. in length in any direction, variations from the standard thickness of 0.02 in. in excess of the allowances above given shall be permitted. For special castings of standard patterns, a variation of 50 per cent greater than allowed for straight pipe shall be permitted.

TABLE I.—VARIATION IN THICKNESS OF SAND-CAST PIPE.

6-IN. PIPE Nominal inside diameter, 6 in. Nominal wall thickness, 0.48 in. Minimum general wall thickness allowed, 0.40 in. Minimum wall thickness limited to 8 in. in any direction allowed, 0.38 in.					8-IN. PIPE Nominal inside diameter, 8 in. Nominal wall thickness, 0.51 in. Minimum general wall thickness allowed, 0.43 in. Minimum wall thickness limited to 8 in. in any direction allowed, 0.41 in.				
Mark	Actual Thickness, in.		Wall Thickness Variation, per cent	Acceptability for Class B	Mark	Actual Thickness, in.		Wall Thickness Variation, per cent	Acceptability for Class B
	Minimum	Maximum				Minimum	Maximum		
6SO1.........	0.39	0.60	54.0	No	8SO1.......	0.41	0.67	63.4	No
6SO2.........	0.39	0.59	51.0	No	8SO2.......	0.36	0.61	69.4	No
6SO3.........	0.46	0.60	30.4	Yes	8SO3.......	0.45	0.53	40.0	Yes
6SO4.........	0.45	0.56	46.6	Yes	8SO4.......	0.46	0.53	15.2	Yes
6SO5.........	0.39	0.53	35.8	No	8SO5.......	0.49	0.56	14.3	Yes
6SN1.........	0.45	0.50	11.2	Yes	8SN1.......	0.45	0.58	27.0	Yes
6SN2.........	0.46	0.52	13.0	Yes	8SN2.......	0.46	0.55	19.5	Yes
6SN3.........	0.49	0.56	14.3	Yes	8SN3.......	0.48	0.58	20.8	Yes
6SN4.........	0.44	0.50	13.6	Yes	8SN4.......	0.50	0.62	24.0	Yes
6SN5.........	0.48	0.62	29.1	Yes	8SN5.......	0.51	0.59	15.7	Yes
6SN6.........	0.45	0.53	17.8	Yes	8SN6.......	0.49	0.56	14.3	Yes

As given in Table 153 on page 24 of the 1925 hand book of the United States Cast Iron Pipe and Foundry Co., entitled "Cast-Iron Pipe," four standard thicknesses are listed for sand-cast pipe. These are known as Classes *A*, *B*, *C*, and *D*, intended for pressures respectively of 43, 86, 130, and 173 lb.

Since these specifications were adopted, centrifugally-cast pipe has been placed on the market. The specifications for centrifugally-cast pipe cast in metal molds, prepared by the United States Cast Iron Pipe and Foundry Co., are given in its 1925 handbook entitled "Cast Iron Pipe" on pages 12 to 15. This kind of pipe is sold in three classes known as Class 50, Class 150 and Class 250. These numbers refer to the allowable pressures. Section 3 in the specification for this pipe discusses allowable variations in thickness as follows:

> The variations allowed above and below the standard thickness as given in the accompanying tables shall not be greater than 0.045 in. for pipe 6-in. in diameter, 0.05 in. for pipe 8-in. in diameter, 0.055 in. for pipe 10-in. in diameter, and 0.06 in. for pipe 12-in. in diameter.

The sand-cast pipe used in these tests was bought by the Detroit Water Board as Class *B* and the centrifugally-cast pipe as Class 150. Our discussion, therefore, in this and in other sections of the report will be on the basis that we are dealing with Class *B* pipe if it is sand cast, and Class 150 pipe if it is centrifugally cast.

TABLE II.—VARIATION IN THICKNESS OF CENTRIFUGALLY-CAST PIPE.

6-IN. PIPE Nominal inside diameter, 6 in. Nominal wall thickness, 0.35 in. Minimum wall thickness allowed, 0.305 in.					8-IN. PIPE Nominal inside diameter, 8 in. Nominal wall thickness, 0.38 in. Minimum wall thickness allowed, 0.335 in.				
Mark	Actual Thickness, in.		Wall Thickness Variation, per cent	Acceptability for Class 150	Mark	Actual Thickness, in.		Wall Thickness Variation, per cent	Acceptability for Class 150
	Minimum	Maximum				Minimum	Maximum		
6CO1	0.39	0.42	7.7	Yes	8CO1	0.44	0.50	13.6	Yes
6CO2	0.34	0.38	11.7	Yes	8CO2	0.42	0.48	14.3	Yes
6CO3	0.41	0.42	2.4	Yes	8CO3	0.40	0.46	15.0	Yes
6CO4	0.37	0.39	5.4	Yes	8CO4	0.42	0.44	4.7	Yes
6CO5	0.38	0.38	0.0	Yes	8CO5	0.43	0.46	7.0	Yes
6CN1	0.39	0.42	7.7	Yes	8CN1	0.46	0.53	15.2	Yes
6CN2	0.33	0.34	3.0	Yes	8CN2	0.39	0.46	18.0	Yes
6CN3	0.36	0.40	11.1	Yes	8CN3	0.38	0.41	7.9	Yes
6CN4	0.42	0.46	9.5	Yes	8CN4	0.40	0.43	7.0	Yes
6CN5	0.35	0.37	5.7	Yes	8CN5	0.42	0.46	9.5	Yes
6CN6	0.37	0.41	10.8	Yes	8CN6	0.40	0.45	12.5	Yes

The variation in thickness findings on the 22 samples of sand-cast pipe are given in Table I. This table shows a wide variation in the wall thickness of the pipe, varying from a minimum of 11.2 per cent to a maximum of 69.4 per cent. Twenty-two per cent of these pipe had thinner walls than is permitted by the 0.08-in. tolerance limit.

The centrifugally-cast pipe showed full control over wall thickness, every pipe examined being well within the allowed variation. The data are recorded in Table II. Not only was excellent control shown in the matter of maintaining wall thickness within the limits of the specifications, but likewise all of the pipe showed a minimum variation in wall thickness, particularly so, if comparison was made with the variation found in the case of the sand-cast pipe. The wall thickness variation, namely, ranged from 2.4 to 18 per cent, which is less than one-third of that found in the sand-cast pipe.

Attention is invited to the importance of control of wall thickness, bearing in mind that it is the minimum thickness that controls in tension, not the nominal. A cursory reading of Section 7 of the American Water Works Association specification, which in part is as follows:

> No pipe shall be accepted, the standard weight of which shall be less than the standard weight by more than 5 per cent, for pipe 16 in. or less in diameter, and 4 per cent for pipe more than 16 in. in diameter, and no excess above the standard weight of more than the given percentage for the several sizes shall be paid for.

might lead one to the conclusion that, since the weight of the pipe is held to a 5 per cent variation, the wall thickness would vary to only a like degree. Examination of Table I shows the fallacy of such a conclusion.

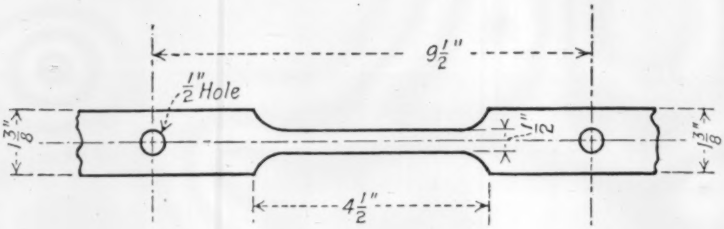


FIG. 1.—Tension Test Specimen.

Engineers who are familiar with foundry operation appreciate that pipe castings cannot be held to a minimum wall thickness variation of 5 per cent. Whether or not more rigid inspection would reduce the wall thickness variation is questionable. Yet the insistent demand of all engineers is for as uniform a material as it is possible to procure, with respect to both properties and dimensions. The centrifugally-cast pipe showed not only superior control over variation in wall thickness, but likewise all of the pipe made in accordance with this process that was examined showed it to be within the wall thickness requirements. In view of these findings, therefore, from a wall thickness standpoint centrifugally-cast pipe is superior to sand-cast pipe.

Tension Tests:

Internal water pressure causes circumferential tension. No standard hydrostatic testing apparatus for pipe was available, so it was finally decided that relative comparisons could be obtained from tension bars taken from sections of the pipe submitted, calculating the bursting strengths from the tension test data on these bars, using minimum wall thickness as the thickness of the section.

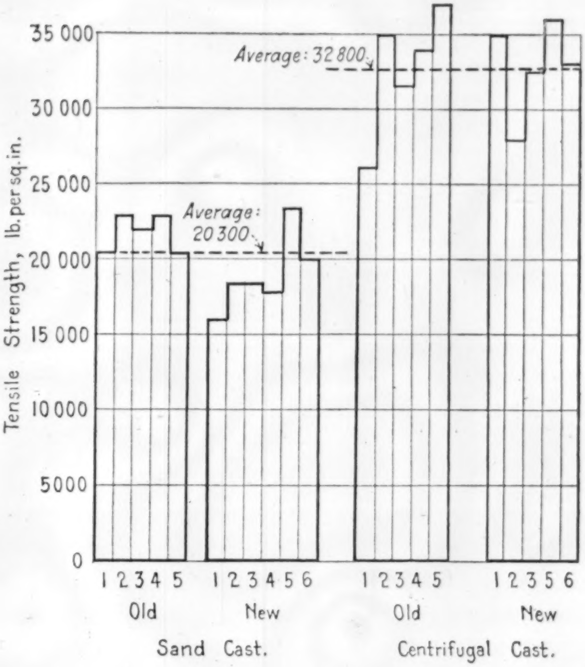


FIG. 2.—Tension Tests of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

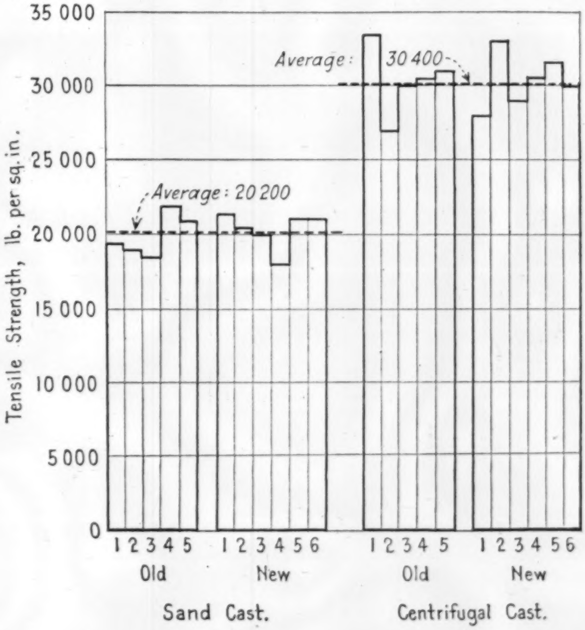


FIG. 3.—Tension Tests of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

Only rough-machined test specimens were cut from the four pieces of pipe tested in the preliminary tests. The relative values of tensile strength are about the same, but the magnitudes are different, most likely due to adverse conditions arising in grips. The average tensile strengths of the 6 and 8-in. centrifugally-cast pipe were 28,000 and 24,260 lb. per sq. in., respectively, while those of the sand-cast pipe for the same sizes were 18,400 and 25,260 lb. per sq. in.

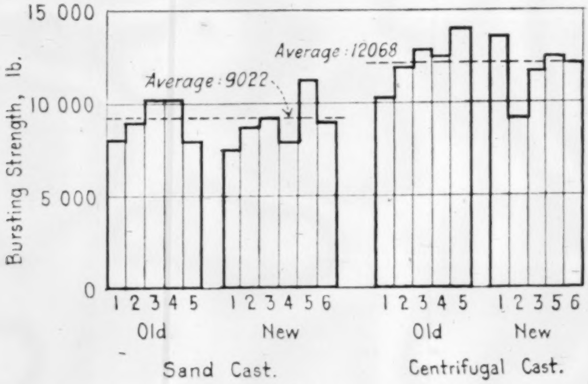


FIG. 4.—Bursting Strength of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

Bursting strength determined by multiplying tensile strength (see Fig. 2) by minimum wall thickness.

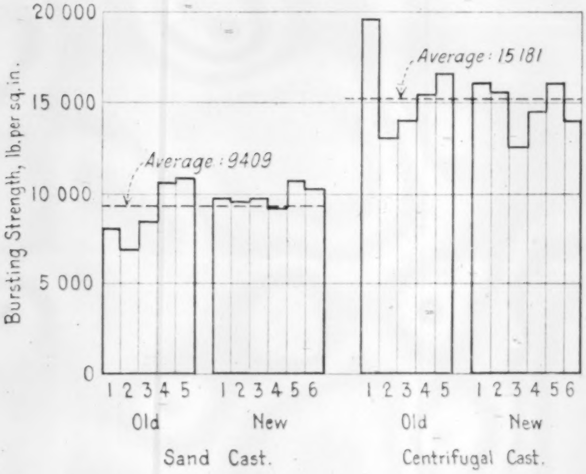


FIG. 5.—Bursting Strength of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

Bursting strength determined by multiplying tensile strength (see Fig. 3) by minimum wall thickness.

Cast iron is recognized as a metal of wide variation in physical characteristics. Such matters as lack of alignment of the test specimens during the application of the load seem to have a greater effect on the results than in the case of ductile materials. Therefore, to remove any possibility of doubt on this point, the tension tests on the 44 samples were made on carefully prepared test specimens, machined in accordance with dimensions given in Fig. 1. One tension test specimen was cut from each of the 6 and 8-in. pipe. The data on these tests are given in Figs. 2, 3, 4 and 5.

The results on the centrifugally-cast pipe are in line with, though slightly under, the figures given in the "Underwriters' Laboratories Report on deLavaud Cast-Iron Underground Water Pipe," dated June 12, 1923, as follows:

	UNDERWRITERS' TESTS	UNIVERSITY OF MICHIGAN TESTS
Average Tensile Strength, lb. per sq. in......	35 658	31 150
Maximum Tensile Strength, lb. per sq. in....	43 580	37 000
Minimum Tensile Strength, lb. per sq. in....	28 960	27 000

Figures 2 and 3 show the unit tensile strengths of 6 and 8-in. pipe, respectively. Figures 4 and 5 give a measure of the bursting strengths of the two sizes of pipe, calculated by multiplying the tensile strength in pounds per square inch by the minimum wall thickness as obtained in the tests for variation in thickness. These results show that from both the standpoint of tension tests and from the standpoint of calculated bursting strengths, the centrifugally-cast pipe is superior to sand-cast pipe.

Using Class *B* sand-cast and Class 150 centrifugally-cast pipe as a basis for comparing the factors of safety of the two sizes of pipe in tension due to internal water pressure, the question becomes one of which pipe is superior. According to the specifications for 6-in. Class *B* sand-cast pipe, the minimum thickness allowable is 0.38 in. for areas less than 8 in. in any length and 0.40 in. for areas greater than 8 in. in any length. The requirement for tensile strength is 20,000 lb. per sq. in. minimum. Figures 2 and 3 show that this average value is attained.

A piece of pipe having a minimum wall thickness of 0.40 in. and a length of 1 in. will stand 0.40 × 1 × 20,000 = 8000 lb. bursting load. The internal pressure which will cause 8000 lb. tensile force is equal to:

$$P = \frac{2 \times 8000}{6} = 2666 \text{ lb. per sq. in.}$$

This calculation is based on the thin cylinder formula and the internal diameter is assumed as a nominal diameter of 6 in. Different pressures would be given by other more complicated cylinder formulas but, comparatively, the values would be no more serviceable than those calculated on the basis of the thin cylinder formula so that all the subsequent calculations are based on it.

On the above basis, the factor of safety is 2666 ÷ 86 = 31, where 86 is the internal pressure allowed by the catalog. If, however, the calculation is not based on the nominal inside diameter, but is based on an inside diameter determined by subtracting twice the minimum wall thickness from the outside diameter, then the internal pressure which will cause a tensile force of 8000 lb. is:

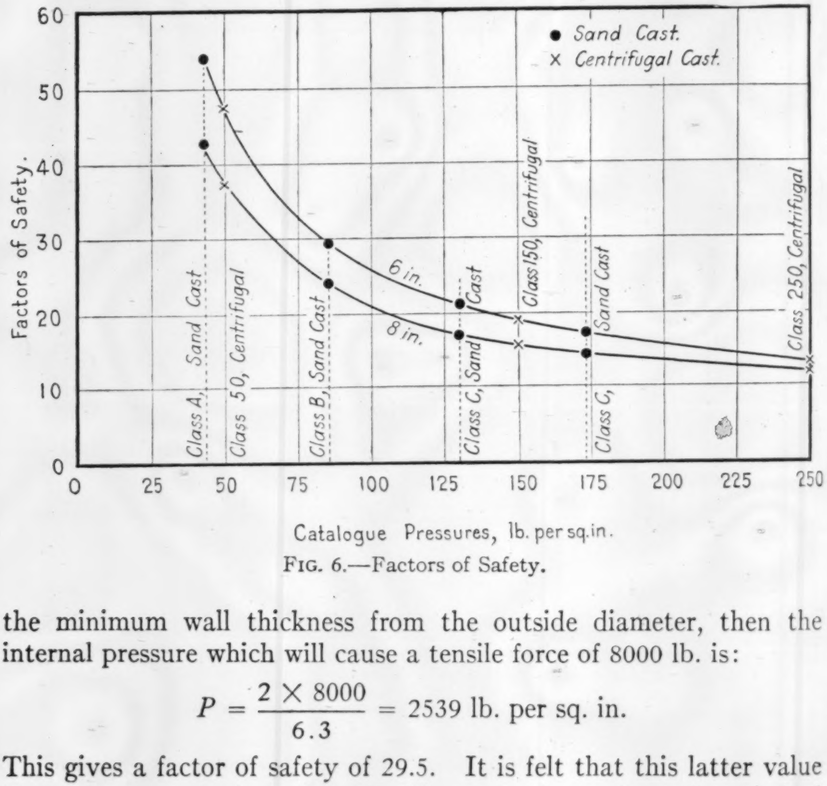


FIG. 6.—Factors of Safety.

$$P = \frac{2 \times 8000}{6.3} = 2539 \text{ lb. per sq. in.}$$

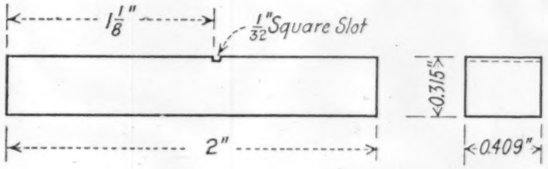
This gives a factor of safety of 29.5. It is felt that this latter value is more representative of the limiting case than where the nominal diameter is used in calculation. For this reason, all subsequent calculations relating to factors of safety will be made on the basis of the actual outside diameter minus twice the minimum wall thickness.

For the 6-in. centrifugally-cast pipe of Class 150, the minimum thickness permissible is 0.305 in. and the minimum stress 30,000 lb. per sq. in. For 1 in. of pipe with an actual outside diameter of 6.9 in.

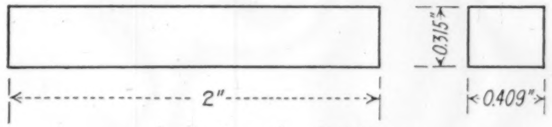
and a minimum wall thickness of 0.305 in., the bursting strength will be 0.305 × 30,000 = 9150 lb. tension. The internal pressure which would cause this load is.

$$P = \frac{2 \times 9150}{6.29} = 2910 \text{ lb. per sq. in.}$$

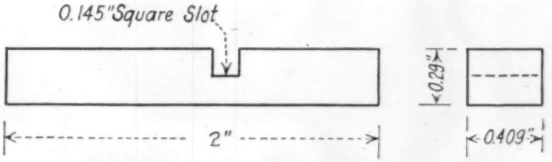
which gives a factor of safety on an actual internal pressure of 150 lb. of 2910 ÷ 150 = 19.4.



Olsen Impact Specimen.
Notched.



Izod Impact Specimen:
No Notch.



"Baby" Olsen Impact Specimen
Notched.

FIG. 7.—Impact Test Specimens.

If centrifugally-cast Class 150 pipe is used on working pressures no greater than 86 lb. per sq. in. as specified for Class *B* sand-cast pipe, the factor of safety is 2910 ÷ 86 = 33.8 for the centrifugally-cast pipe, which is slightly more than the factor of safety for Class *B* sand-cast pipe.

There is a possibility of an engineer at first concluding that the centrifugally-cast pipe has been designed with lower factors of safety than are used on sand-cast pipe, if the 29.5 factor of safety for the sand-cast pipe at 86 lb. per sq. in. pressure is compared with the 19.4

factor of safety for the centrifugally-cast pipe at a pressure of 150 lb. per sq. in. However, an examination of the factors of safety for the various classes of sand-cast and centrifugally-cast pipes, based on their rated working pressures, will show that the factors of safety decrease as a function of the increase in the allowable working pressure. That is, 6-in. Class *A* sand-cast pipe figures out to have a factor of safety of 54.2, Class *B* a factor of safety of 29.5, Class *C*, 21.2, and Class *D*, 17.6. For the 6-in. centrifugally-cast pipe, the 50-lb. class has a calculated factor of safety of 47.9; 150-lb. class, 19.4; and the 250-lb. class, 13.3. These factors have been plotted in Fig. 6 and it will be seen that the values for the centrifugally-cast pipe fall almost on the same curve as the values for the sand-cast pipe. The same is true of the 8-in. pipes, as will be noted from this same chart, so that on a basis of these curves it can be said that the factors of safety for the centrifugally-cast pipe are comparable with those for the sand-cast pipe.

Shock or Impact Tests:

Impact tests afford a means of measuring the comparative resistance of metals to shock or impact. There are several different types of machines on which those tests may be made. In general, they vary in the shape of the test specimen and in the manner of holding it. Regardless of make, all impact testing machines consist of three principal parts, a moving mass having a known kinetic energy at the instant the blow is struck, an anvil on which the specimen is mounted, and a device for measuring the kinetic energy in the moving mass after the specimen has been ruptured.

The results from one machine will not necessarily agree with those from another machine of a different type. For that reason, tests were made with three different types of machine, using both notched and unnotched specimens. The machines used were the Izod, the Olsen and the "Baby" Olsen. Figure 7 shows the type and size of specimens used in each of these machines.

Because of the fact that the 100 ft-lb. impact machines are far greater in capacity than necessary when testing cast iron, which has but relatively slight resistance, accurate readings of resistance are not obtainable. By drawing the pendulum back to a point such that it only had 20 ft-lb. of potential energy, very consistent results were secured, which, while not in any way related to the results obtained with 100 ft-lb. of potential energy, gave what seemed to be consistent and relative values between the different kinds of cast iron in this study.

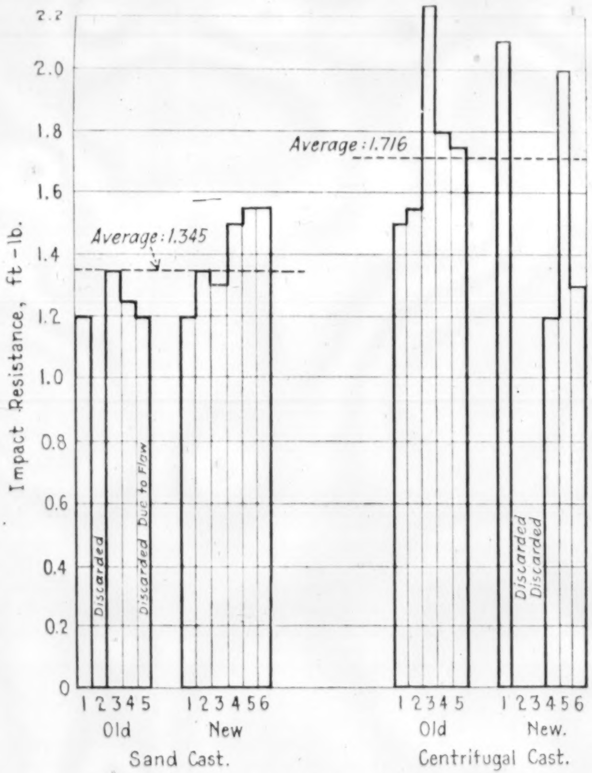


FIG. 8.—Standard Izod Impact Tests of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.315 by 0.409 in.
No notch. Blow on outside surface of pipe.

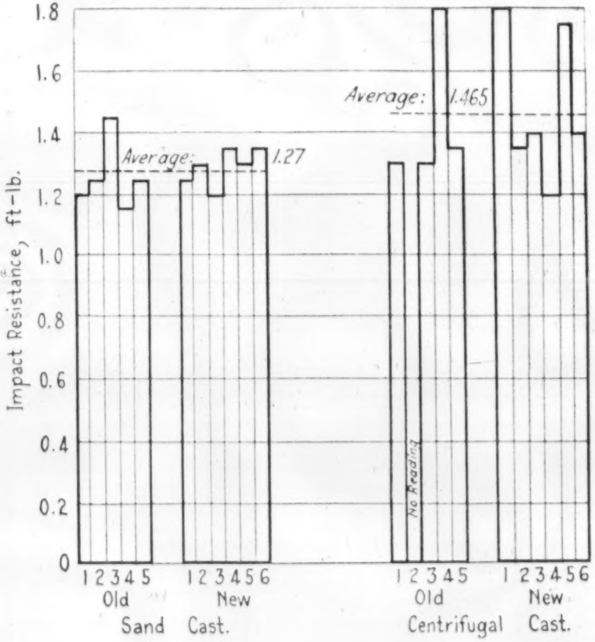


FIG. 9.—Standard Izod Impact Tests of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.315 by 0.409 in.
No notch. Blow on outside surface of pipe.

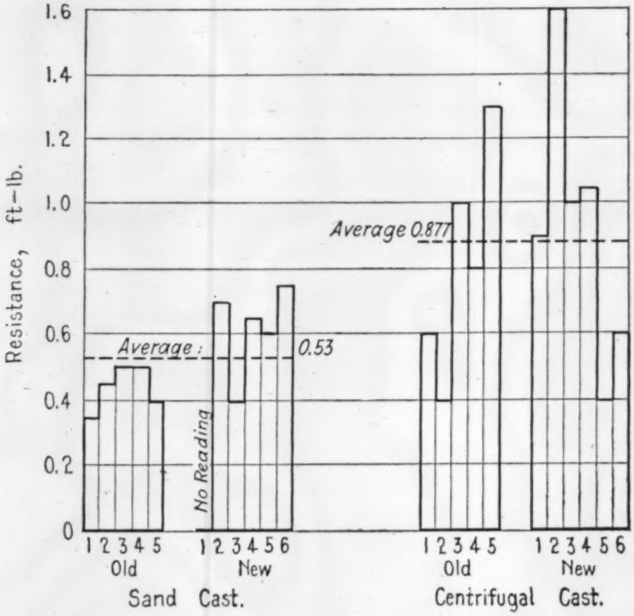


FIG. 10.—Standard Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe. Size of specimen 0.315 by 0.409 in. No notch. Blow on outside surface of pipe.

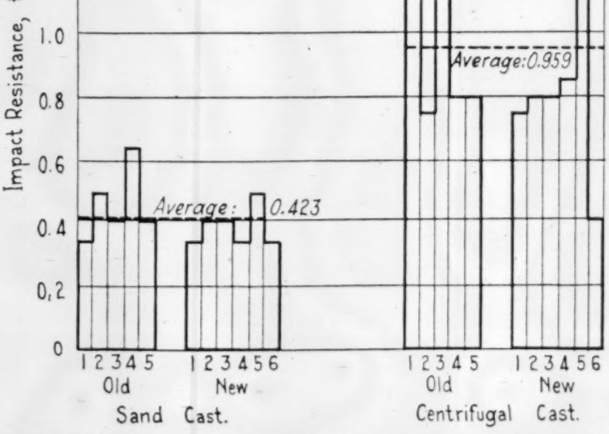


FIG. 11.—Standard Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe. Size of specimen 0.315 by 0.409 in. No notch. Blow on outside surface of pipe.

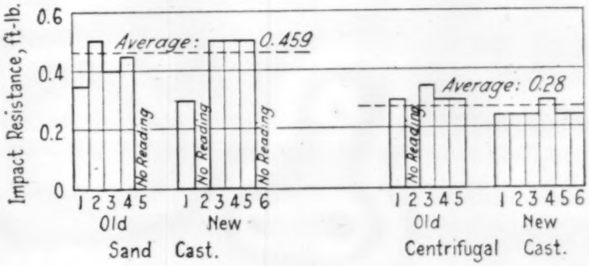


FIG. 12.—Standard Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe.
Size of specimen 0.315 by 0.409 in. Notched. Blow on outside surface of pipe.

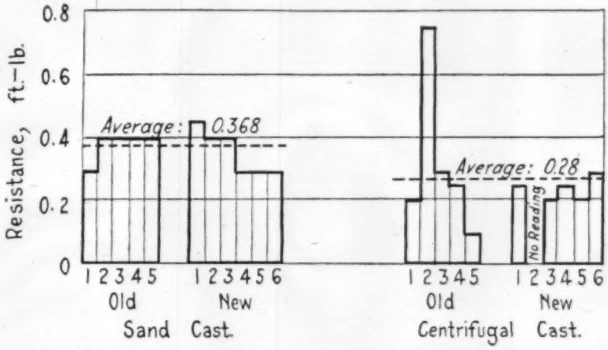


FIG. 13.—Standard Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe.
Size of specimen 0.315 by 0.409 in. Notched. Blow on outside surface of pipe.

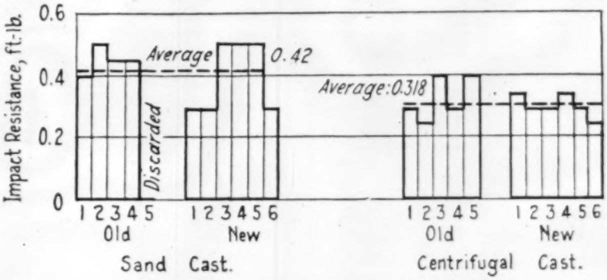


FIG. 14.—Standard Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe.
Size of specimen 0.315 by 0.409 in. Notched. Blow on inside surface of pipe.

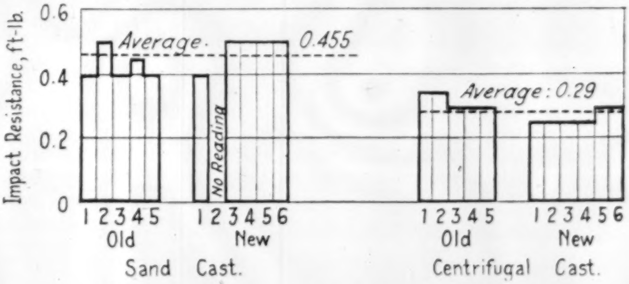


FIG. 15.—Standard Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe.
Size of specimen 0.315 by 0.409 in. Notched. Blow on inside surface of pipe.

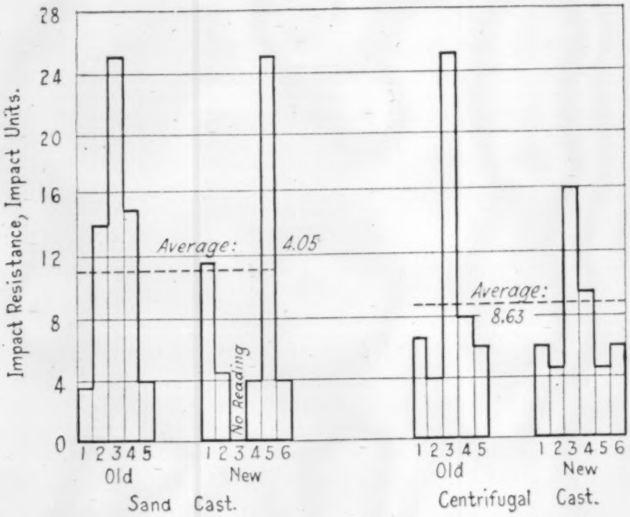


FIG. 16.—"Baby" Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.105 by 0.409 in.
No notch.

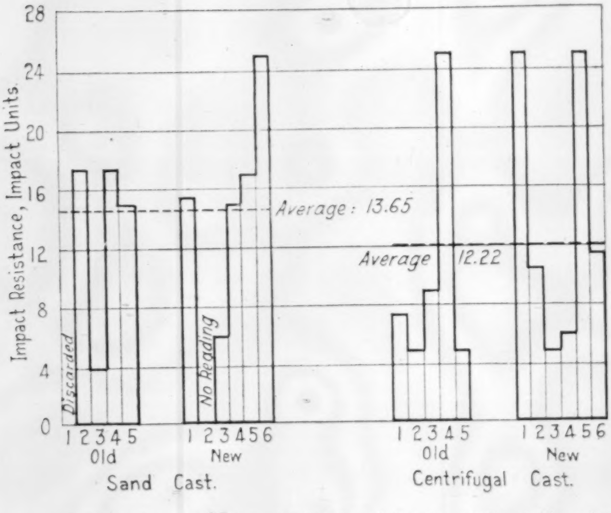


FIG. 17.—"Baby" Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.105 by 0.409 in.
No notch.

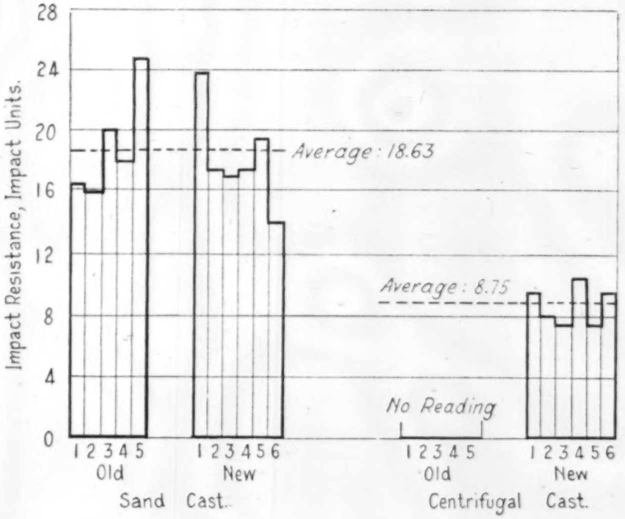


FIG. 18.—"Baby" Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.29 by 0.409 in.
Notched in inside surface of pipe.

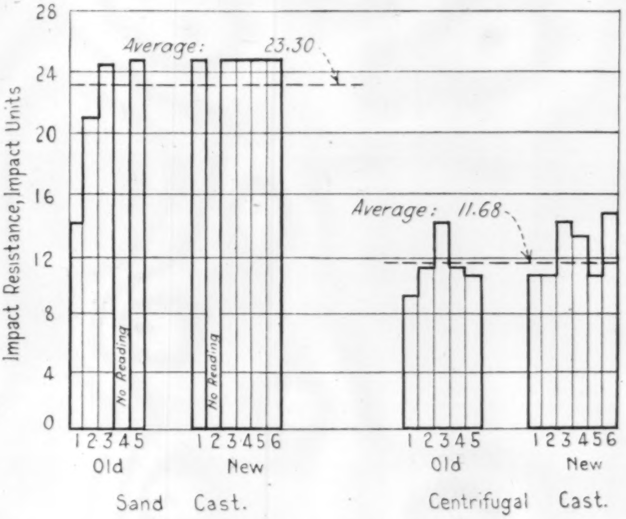


FIG. 19.—"Baby" Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

Size of specimen 0.29 by 0.409 in.
Notched in inside surface of pipe.

The unnotched specimens for each machine were made the same size as the notched specimens, except in the case of the "Baby" Olsen.

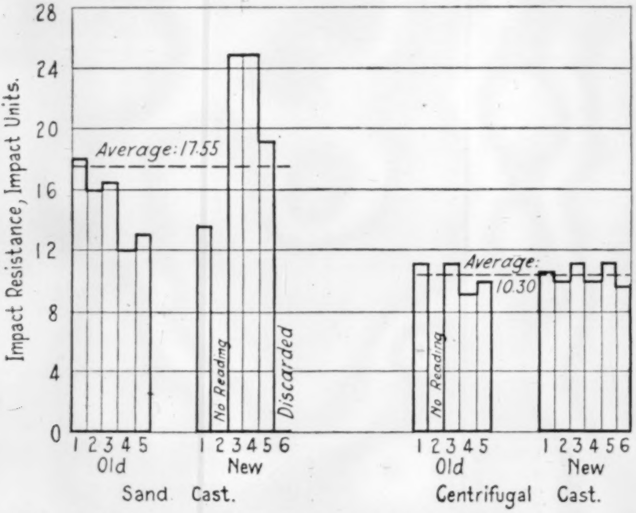


FIG. 20.—"Baby" Olsen Impact Test of 6-in. Sand-Cast and Centrifugally-Cast Pipe. Size of specimen 0.29 by 0.409 in. Notched in outside surface of pipe.

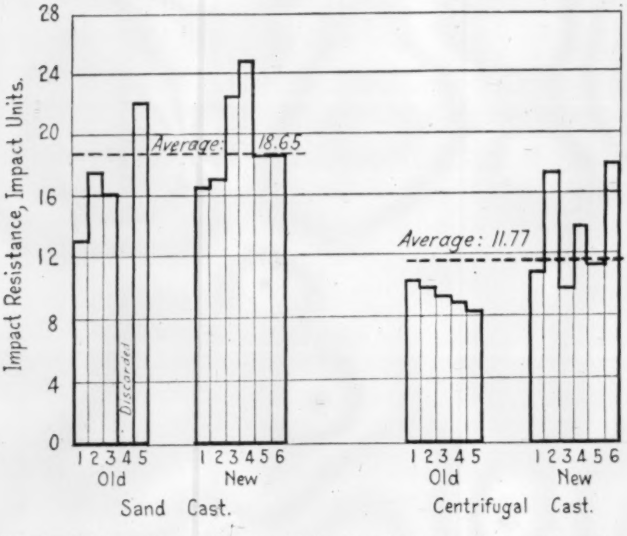


FIG. 21.—"Baby" Olsen Impact Test of 8-in. Sand-Cast and Centrifugally-Cast Pipe. Size of specimen 0.29 by 0.409 in. Notched in outside surface of pipe.

It was found that the latter machine did not have sufficient energy capacity to break the unnotched specimens and the specimens were therefore made 0.04 in. thinner than the thickness of the notched specimens at the notched section.

It was felt that because of the method of casting, the impact resistance of the centrifugally-cast pipe specimens would vary depending on whether the notch was in the inside or the outside portion of the pipe. The iron immediately in contact with the metal wall of the mold would have a greater chill than the inside metal and, therefore, should have somewhat different properties. For this reason, tests on the Standard and "Baby" Olsen were made with the notched cut on the inside of some samples and on the outside of others.

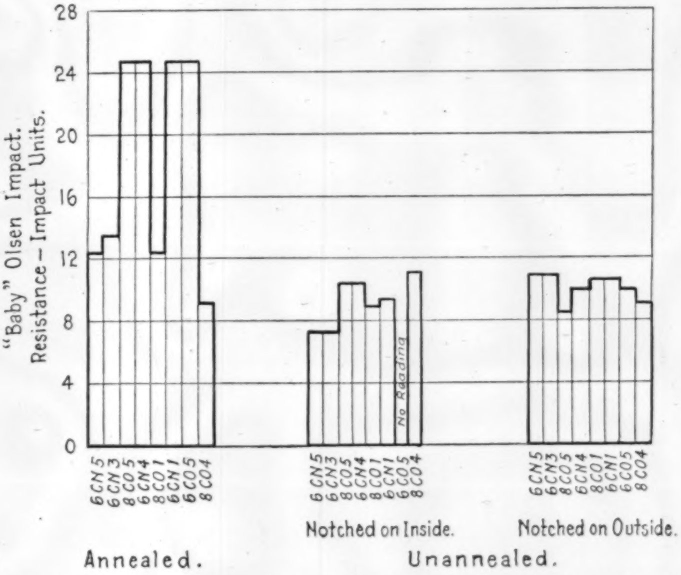


FIG. 22.—Effect of Annealing on Impact Test Values of 6 and 8-in. Centrifugally-Cast Pipe.

"Baby Olsen" Impact Test.
Size of specimen 0.145 by 0.409 in.
Notched.

Some of the notched specimens cut from the centrifugally-cast pipes for the "Baby" Olsen test were annealed and then broken.

The results of the tests using the Izod machine are plotted in Figs. 8 and 9. The results of the tests on the Standard Olsen machine are shown in Figs. 10, 11, 12, 13, 14, and 15. The results from the tests on both of these machines favor the centrifugally-cast pipe, considering the unnotched pipe specimens, and the sand-cast pipe considering the notched specimens.

The results of the tests made on the "Baby" Olsen machine are plotted in Figs. 16, 17, 18, 19, 20, and 21. These results favor the sand-cast pipe, considering either the notched or unnotched specimens, though in the latter case the difference is slight.

The results of the annealed specimens of centrifugally-cast pipe in the "Baby" Olsen machine are given in Fig. 22. The results on the corresponding unannealed pieces taken from Figs. 18, 19, 20 and 21 are recorded in Fig. 22 for comparison. Annealing measurably increased the impact resistance of the centrifugally-cast pipes.

Radial Compression Test:

The radial compression test consisted in compressing radially, complete pieces of pipe 1 ft. long in the manner shown in Fig. 23.

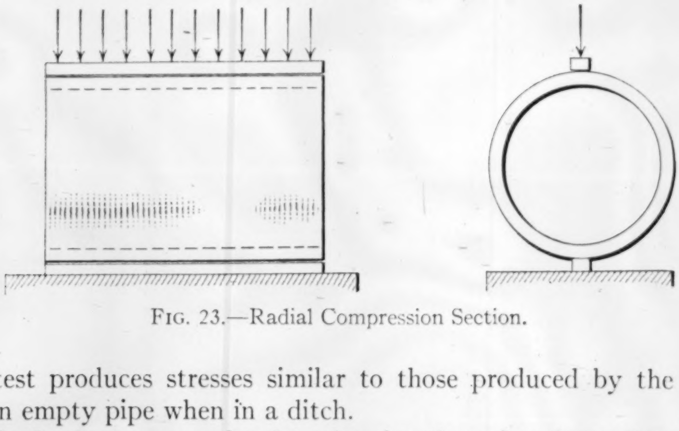


FIG. 23.—Radial Compression Section.

This test produces stresses similar to those produced by the earth load on empty pipe when in a ditch.

The load was transmitted to the pipe through soft wood cushions on top and bottom, to insure reasonably even distribution. Load increments of 500 lb. were put on and corresponding inside deflection was read to 0.001 in.

The radial crushing load results are shown in Figs. 24 and 25. Maximum radial deflections are given in Figs. 26 and 27.

Under the radial compression test the 6-in. sand-cast pipe withstands a 20 per cent greater load than the centrifugally-cast pipe. In the case of the 8-in. pipe, however, the difference to the credit fo the sand-cast pipe is but 5 per cent.

The radial deflection results, as shown in Figs. 26 and 27 favor the centrifugally-cast pipe. The extent to which any weight should be placed on the findings is questionable. It is true that the greater deflection is an indication that the centrifugally-cast pipe is more elastic, possibly more ductile. Or again, the higher reading for the

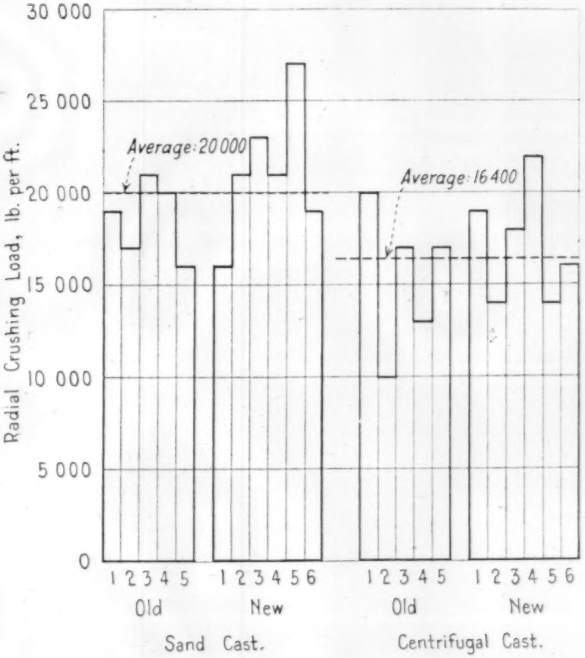


FIG. 24.—Radial Compression Tests of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

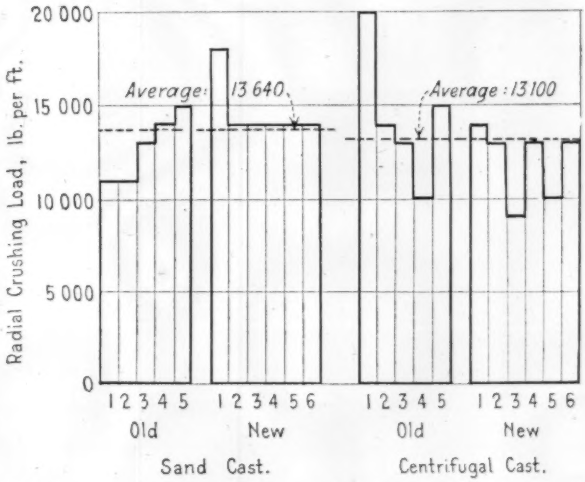


FIG. 25.—Radial Compression Tests of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

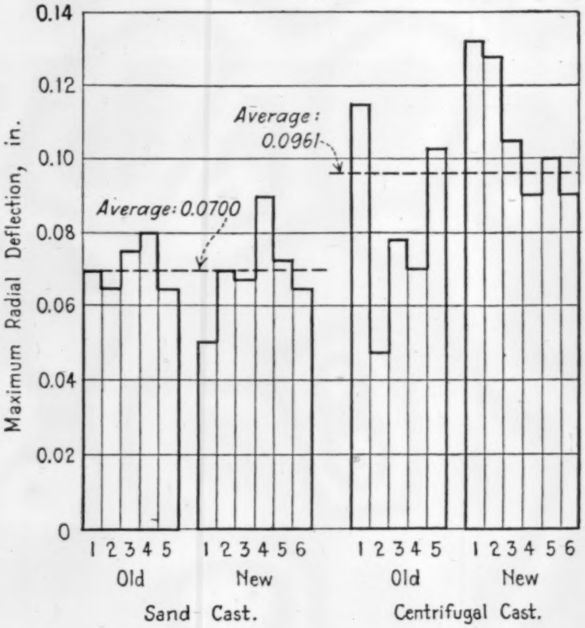


FIG. 26.—Radial Compression Tests of 6-in. Sand-Cast and Centrifugally-Cast Pipe.

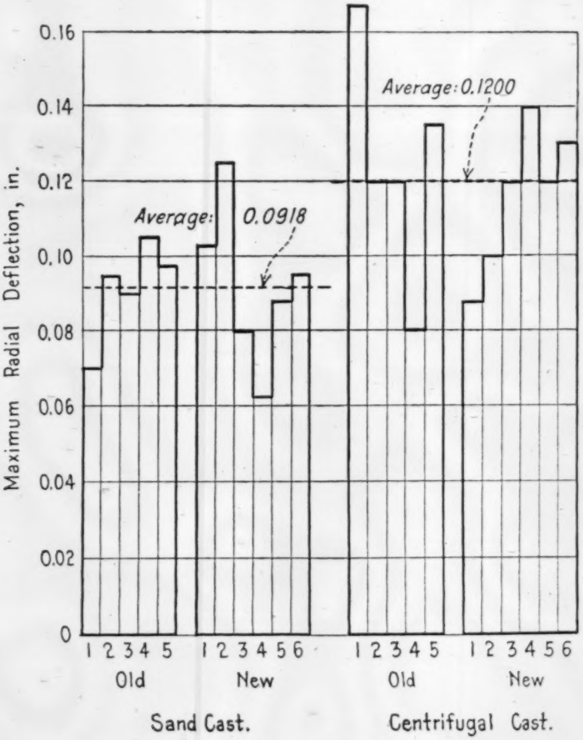


FIG. 27.—Radial Compression Tests of 8-in. Sand-Cast and Centrifugally-Cast Pipe.

centrifugally-cast pipe may result from the fact that the pipe is thinner. There are some circumstances where a yielding pipe may be helpful and certainly some which would demand a stiffer pipe with a greater resistance to bending. So the greater radial deflection of the centrifugal pipe does not necessarily indicate a superior pipe as it lies in the ground in service.

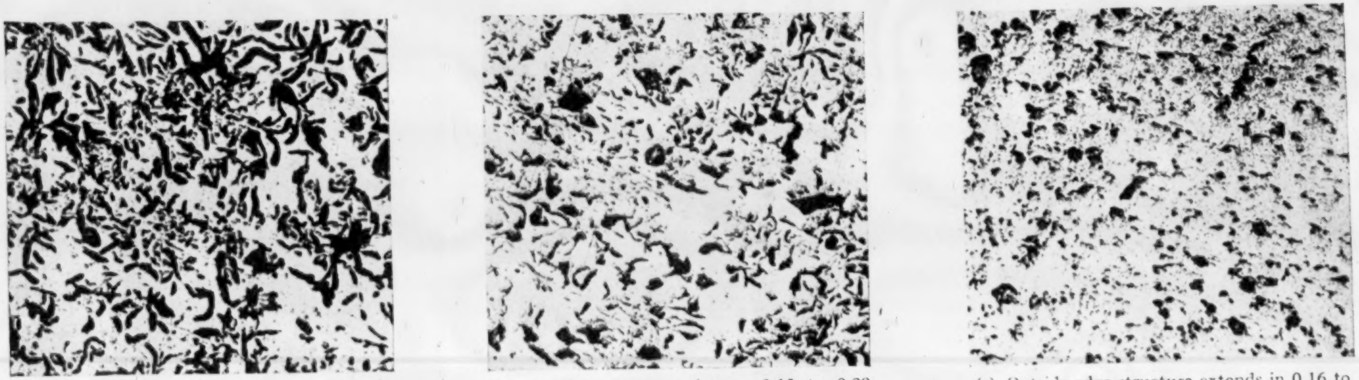
Removal of Strains:

The following statements are found in the Underwriters' Laboratories "Report on deLavaud Cast-Iron Underground Water Pipe," dated June 12, 1923:

> It is well known to engineers that in sand-casting, cooling strains and other strains are invariably present. The amount of such strains it is impossible to estimate, nor is it feasible to eliminate these strains. The fact of the strains is well proved by the unexplained breaks in pipe lines showing perfect metal at the place of rupture and also the fact that frequently a sand-cast pipe cracks, and then when pressure is removed the crack closes up, making it almost impossible to detect. These strains may result from the composition of the iron, the method of casting, the condition of the sand, the manner of cooling, the turning out of the pipe, whether hot or cold, and atmospheric conditions at the time of such turnout, or at the time of casting.
>
> We claim that the deLavaud pipe, by reason of its annealing and method of manufacture, eliminates these strains. This is proved by the complete rupture of the pipe under pressure, and by the fact that when it is cracked the crack remains open, and does not close when the pressure is removed.

The annealing which is given to metal-mold centrifugally-cast pipe should free it from all casting strains. In this respect, therefore, centrifugally-cast pipe is superior to sand-cast pipe because little, if any, sand-cast pipe is annealed and the cooling conditions surrounding certain of the sand-cast pipe cannot help but set up strains.

But it should be borne in mind that it is the annealing and not the casting process that does away with cooling strains and that if not carefully done the strains are apt to be worse in centrifugally-cast pipe than in sand-cast pipe. It should also be borne in mind that in the first four samples of pipe submitted, there was one cracked centrifugally-cast pipe which was not discovered until put into the milling machine for cutting specimens. The crack had evidently been in the pipe before it was dipped, for the solution had penetrated into the crack slightly. The crack could have been a pure shrinkage crack or impact crack or, what is more probable, a combination of the two. So under "Removal of Strains" the practice, if thoroughly carried out, should favor the centrifugally-cast, but the fact remains that the only specimen of those submitted in this test which displayed evidence of a cooling strain was a centrifugally-cast pipe.

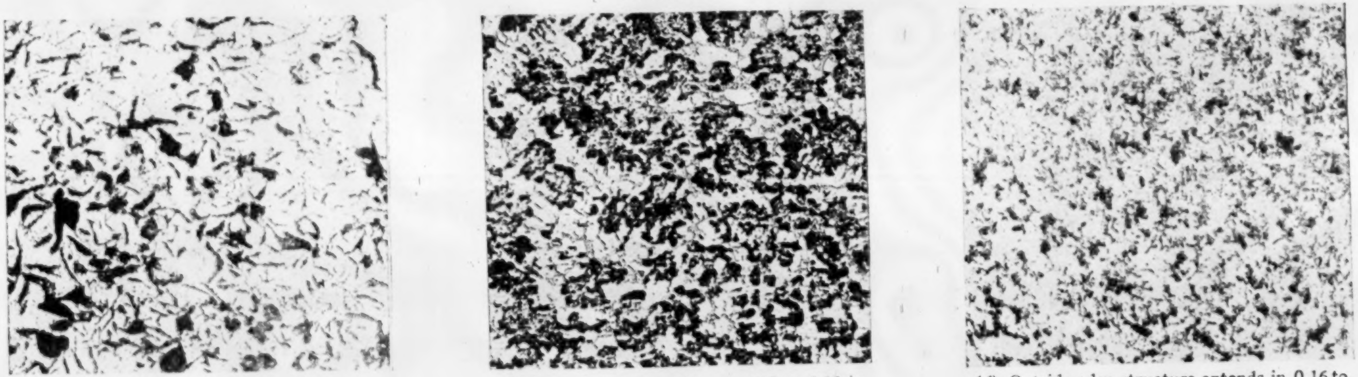


(*a*) Inside edge structure decarbonized for 0.01 in.

(*b*) Central structure changes 0.15 to 0.20 in. from inner surface.

(*c*) Outside edge structure extends in 0.16 to 0.20 in. from outside surface.

Unetched



(*d*) Inside edge structure decarbonized for 0.01 in.

(*e*) Central structure changes 0.15 to 0.20 in. from inner surface.

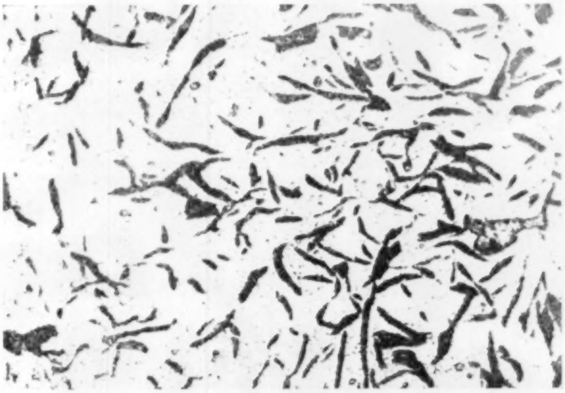
(*f*) Outside edge **structure extends** in 0.16 to 0.20 in. from outside surface.

Etched

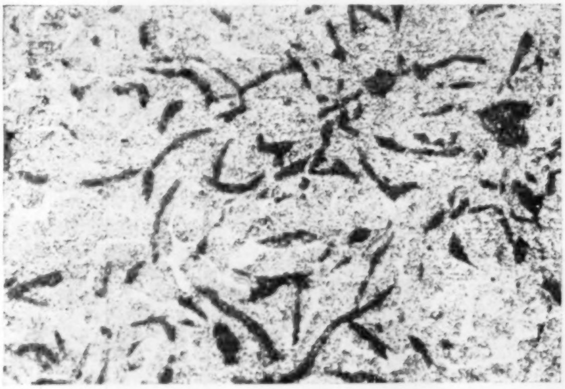
FIG. 28.—Photomicrographs of Centrifugally-Cast Pipe (× 100).

Inside Smoothness:

The inside smoothness of the first four pipes investigated favored the centrifugally cast, but one or two samples of the centrifugally-cast pipe were so rough as to offset any advantage gained by the general run. However, of the 44 pipe later investigated, chosen at random, 98 per cent of the centrifugally-cast pipe were smoother on the inside.



Unetched



Etched

FIG. 29.—Photomicrographs of Sand-Cast Pipe (× 100).

This is of some importance in reducing the friction coefficient, and with the possible greater diameter of the centrifugally-cast pipe will make possible the delivery of more water for any given pressure. For this reason, therefore, there would appear to be an advantage to centrifugally-cast pipe.

Cutting Tests:

No actual tests were made at the University of Michigan to determine the cutting characteristics of the two classes of pipe.

From reports received from workmen on Detroit installations, we have been informed that the centrifugally-cast pipe cuts more satisfactorily than the sand-cast pipe. Accepting these reports as evidence, the advantage from this characteristic would appear to be on the side of the centrifugally-cast pipe.

Metallographic Examination:

The metallographic study of the sand-cast and the centrifugally-cast pipe discloses the differences one would expect to find. In either pipe one should find graphite, ferrite, or pure iron, and the combined carbon-containing constituent. All of these constituents are visible in the etched samples, though only graphite and background are in evidence in the unetched samples. One would expect to find smaller sized graphite particles in the centrifugally-cast iron than in the sand-cast iron and this is the case.

Eight photomicrographs are submitted in Figs. 28 and 29, six from centrifugally-cast iron and two from sand-cast iron. Four are unetched and four are etched. In the photomicrographs of the unetched samples, the black is the graphite and the white is ferrite or the carbon-containing constituent. In the photomicrographs of the etched samples, the deep black is the graphite, the black half-tone is the carbon-containing constituent, and white is the ferrite.

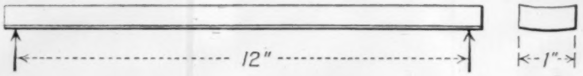


FIG. 30.—Transverse Test Specimen.

The six samples of centrifugally-cast iron were chosen for the purpose of showing the difference in structure that occurs from the outside to the inside surface of the pipe. Note that in both the etched and unetched sections taken from the outside edge, the graphite is quite finely distributed and is of globular type. The photomicrographs taken from the central sections show the graphite to be in the form of plates or flakes, although these plates or flakes are much smaller than the plates or flakes found in the sand-cast iron. The photomicrographs taken from the inside surface of the centrifugally-cast pipe show a section about 0.01 in. thick virtually free from graphite, but beyond that the graphite is similar to that found in the central section of the pipe.

Attention is likewise called to the fact that the combined carbon in the centrifugally-cast pipe is much less than the combined carbon in the sand-cast pipe. The photomicrographs show this to be the case because of the almost complete absence of the half-tone structure, and our chemical analyses corroborate this finding. We would call attention to the fact that there does not appear to be a gradual transition from the flake or plate graphite formation in the case of the centrifugally-cast iron to the globular formation. The transition is sharp. In most samples examined, this zone occurs closer to the inside than it does to the outside. The photomicrographs shown in Figs. 28 (*b*) and (*e*) have been selected for the specific purpose of showing the transition just mentioned.

TABLE III.—RESULTS OF TRANSVERSE TESTS ON SECTIONS FROM CENTRIFUGALLY AND SAND-CAST PIPE.

SPECIMEN	LOADED	LOAD, LB.	DEFLECTION, IN.
6-in. Centrifugal	Concave side up	480	0.32
6-in. Centrifugal	Concave side up	460	0.38
6-in. Centrifugal	Concave side down	400	0.36
Average		447	0.35
6-in. Sand Cast	Concave side up	650	0.20
6-in. Sand Cast	Concave side up	600	0.23
6-in. Sand Cast	Concave side down	475	0.28
Average		575	0.237
8-in. Centrifugal	Concave side up	700	0.19
8-in. Centrifugal	Concave side down	650	0.24
8-in. Centrifugal	Concave side up	700	0.19
Average		683	0.21
8-in. Sand Cast	Concave side up	780	0.29
8-in. Sand Cast	Concave side down	860	0.26
8-in. Sand Cast	Concave side up	820	0.28
Average		820	0.28

TABLE IV.—BRINELL HARDNESS TESTS.

		BRINELL READING	
MARK	SIZE OF PIPE, IN.	SAND CAST	CENTRIFUGALLY CAST
—	6	187	215
—	8	192	241
O3	6	163	187
N3	6	153	191
O3	8	187	205
N3	8	187	196
Average		178	206

Transverse Tests:

Transverse tests were made on samples of each of the four pieces of pipe examined previous to the group of 44. The shape of the test specimen is given in Fig. 30. The results of these tests are given in Table III.

These results favor the sand-cast pipe from the standpoint of resistance to transverse loads and the centrifugally-cast pipe from the standpoint of deflection before breaking. In each size and kind, two specimens were set concave side up and one convex side up so that the results are comparative and conclusive, checking the radial compression tests in both resistance and deflection, that is, the sand-cast pipe resists greater loads and is stiffer. As was mentioned in the radial deflection discussion, the greater deflection for the centrifugally-cast pipe may be due to greater elasticity, greater ductility, or merely thinner sections—whatever the reason, the facts bespeak themselves.

Hardness Tests:

Table IV gives a record of the Brinell hardness determinations made in the University of Michigan laboratories.

It would appear from these tests that the centrifugally-cast pipe is slightly harder than sand-cast pipe. This is doubtless true, but a Brinell hardness up to 220 causes no machining difficulties and therefore the higher hardness of the pipe can not be classed as objectionable from that standpoint. It contributes, however, to the objectionable brittleness qualities of any cast-iron pipe and checks the inferior impact values of the centrifugally-cast pipe.

Chemical Composition:

The United States Cast Iron Pipe and Foundry Co. have submitted the following as a typical chemical analysis of deLavaud centrifugally-cast iron pipe:

Total Carbon, per cent	3.50
Graphite Carbon, per cent	3.40
Combined Carbon, per cent	0.10
Silicon, per cent	2.10
Manganese, per cent	0.55
Sulfur, per cent	0.06
Phosphorus, per cent	0.78

Analyses of this same class of pipe made in the laboratories of the University of Michigan show the following:

	6 IN.	6CN3	6CO3	8 in.	8CN3	8CO3
Total Carbon, per cent	3.58	3.61	3.62	3.47	3.66	3.62
Graphite Carbon, per cent	3.49	3.17	3.18	3.31	3.20	3.14
Combined Carbon, per cent	0.09	0.44	0.44	0.16	0.46	0.37
Silicon, per cent	2.32	1.91	1.84	3.27	1.93	2.43
Manganese, per cent	0.38	0.40	0.51	0.56	0.43	0.50
Sulfur, per cent	0.052	0.039	0.029	0.038	0.037	0.040
Phosphorus, per cent	0.636	0.750	0.767	0.626	0.775	0.735

Analyses of sand-cast pipe made in the University laboratories follow:

	6 in.	6SN3	6SO3	8 in.	8SN3	8SO3
Total Carbon, per cent	3.78	3.72	3.65	3.67	3.62	3.70
Graphite Carbon, per cent	3.17	2.86	2.77	3.05	2.67	2.75
Combined Carbon, per cent	0.61	0.86	0.88	0.62	0.95	0.95
Silicon, per cent	2.16	1.61	1.85	2.39	1.60	1.54
Manganese, per cent	0.79	0.46	0.54	0.45	0.48	0.52
Sulfur, per cent	0.084	0.061	0.078	0.071	0.047	0.053
Phosphorus, per cent	0.366	0.807	0.680	0.605	0.646	0.650

In the "Underwriters' Laboratories Report on deLavaud Cast-Iron Underground Water Pipe" dated June 12, 1923, there are given on page 19 the results obtained from nine analyses of iron, from which the pipe used in the Strength of Pipe Tests were made, were as follows:

	SILICON, PER CENT	SULFUR, PER CENT	MANGANESE, PER CENT	PHOSPHORUS, PER CENT
Average	2.53	0.077	0.63	0.74
Maximum	2.73	0.114	0.72	0.80
Minimum	2.35	0.060	0.54	0.68

A report of analyses in "A Departure in Pipe Foundry Practice," by Peter Gillespie in the Journal of the American Water Works Association, Vol. 9, No. 5, September, 1922, page 706, follows:

	MACHINE-MADE PIPE	SAND-CAST PIPE
Carbon, per cent	3.45	3.67
Manganese, per cent	0.49	0.61
Sulfur, per cent	0.053	0.044
Phosphorus, per cent	0.563	0.654
Silicon, per cent	2.48	2.00

These analyses show the following range in composition:

	DELAVAUD CENTRIFUGALLY-CAST PIPE		SAND-CAST PIPE	
	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM
Total Carbon, per cent	3.47	3.66	3.62	3.78
Graphite Carbon, per cent	3.14	3.49	2.67	3.17
Combined Carbon, per cent	0.09	0.46	0.61	0.95
Silicon, per cent	1.84	2.73	1.54	2.39
Manganese, per cent	0.38	0.76	0.45	0.79
Sulfur, per cent	0.029	0.114	0.047	0.084
Phosphorus, per cent	0.626	0.800	0.366	0.807

All of these analyses show well-controlled foundry practice. The lower combined carbon content in the centrifugally-cast pipe explains the easier cutting qualities of the pipe as stated by those who use the pipe in the field.

CONCLUSIONS

Ten separate types of tests were made on 48 pieces of pipe, 24 of which were centrifugally cast and 24 of which were sand cast. From these ten different types of tests, reports have been made on twelve properties. Four of these favor sand-cast pipe, two are neutral, and six favor the centrifugally-cast pipe.

These twelves properties have been divided into five of major and seven of minor importance, or of greater and lesser weight. Of the five major tests, each type gets favored under two tests and one test seems questionable. Of the minor tests the centrifugal is favored by four and the sand cast by two.

IMPORTANCE OF TESTS	TESTS	FAVOR
Primary	Tension	Centrifugal
Primary	Variation in Thickness	Centrifugal
Primary	Removal of Strain	No decision
Primary	Impact	Sand Cast
Primary	Radial Compression	Sand Cast
Secondary	Radial Deflection	Centrifugal
Secondary	Transverse	Sand Cast
Secondary	Inside Smoothness	Centrifugal
Secondary	Cutting	Centrifugal
Secondary	Metallographic	Centrifugal
Secondary	Hardness	Sand Cast
Secondary	Chemical	No decision

The tension tests, variation in thickness, and removal of strains are placed first, because all have to do with the most necessary thing of withstanding the bursting pressure of ordinary service and overload.

Variation in thickness comes in because the total resistance is a function of the thickness, as well as of the quality of the pipe. Strictly a speaking, variation, soar as influence on tensile strength is concerned, should and would disappear if pipe were delivered strictly up to specification as to minimum thickness. This is not the case, however, so far as our records show. Thickness variation, therefore, becomes a matter of real importance. In a number of the sand-cast pipe examined, for example, the variation in thickness was well toward 50 per cent and in one case it amounted to 63 per cent. The centrifugally-cast pipe, on the other hand, showed a maximum variation of 18 per cent.

Removal of strains is important in that if the pipe is under a circumferential stress, due to cooling, this stress will by just that much reduce the service stress which the pipe will be able to withstand.

Impact resistance is given a place of importance because of the well-known tendency of cast iron to lack toughness. There is not as much impact in the service function of the pipe, but previous to putting pipe into service it is subject to considerable impact, and breakage forms a considerable item in the cost of a pipe line.

A high radial compression resistance is important but probably not so much as the first three items, particularly the tension test, which has so much to do with resistance to bursting. Pipe is always under tension throughout its length and that is the real reason for placing tension tests first. Even more so when not delivering water, every length or fitting, regardless of its location in the pipe system, is under tension. Considering the high resistance to compression, ordinary depths of ditch do not produce a great deal of load on a pipe, but occasionally under a railway crossing or similar place radial resistance does come into play.

The remaining tests are of secondary importance in this particular investigation. They were necessary in that in almost every case they form supporting evidence to the conclusions arrived at in those tests designed to simulate service conditions. For instance, the metallographic tests show that the tensile strength of the centrifugally-cast pipe should be superior.

Radial deflection probably favors centrifugally-cast pipe and is of some importance. It means that in case a pipe was forced to bend it would bend further than would sand-cast pipe and in some cases, at least, this would relieve the bending force.

The transverse test is of lesser importance and is largely a confirmation of the radial compression tests.

Inside smoothness is a factor worth considering, which, when combined with the actual fact of a slightly greater inside diameter to the centrifugal pipe gives greater water delivering capacity to the centrifugally-cast pipe.

These findings lead us to the following conclusions:

1. With present practice and on the basis of similar costs, we find no outstanding advantage to either the centrifugally-cast or sand-cast pipe.

2. With present practice and on the basis of lower costs for the centrifugally-cast pipe, we would favor the purchase of centrifugally-cast pipe.

DISCUSSION

Mr. Ellis. MR. A. R. ELLIS.[1]—I should like to ask the authors of the paper whether this sand-cast pipe is of European manufacture or American manufacture? A great deal of cast-iron pipe of European manufacture is used at Detroit.

Mr. Menefee. MR. F. N. MENEFEE.[2]—According to information from the City authorities in Detroit, none of this pipe was of European manufacture.

Mr. Richart. MR. F. E. RICHART.[3]—The subject of this paper is one of great interest at the present time. I should like to discuss one or two features of the paper, particularly the use of the tension test of a small specimen cut from the pipe wall as a basis on which to calculate the bursting strength of the pipe. In the paper by Mr. Talbot and myself,[4] to which the authors have made reference, studies were reported upon of the relation between the tensile strength of small specimens cut longitudinally from the pipe wall and the actual bursting strength of the pipe. It was found that the two strengths differed consistently, the strength of the small specimen being considerably greater than that of the pipe. This is to be expected, since the bursting test seeks out the weakest point in the pipe and further, the small specimen may be expected to be more nearly free from initial strains than the full section of the pipe. In the tests mentioned, the ratio of the average tensile strength for two types of small tension specimens to the bursting strength of the 12-ft. sections of pipe was 1.56 for sand-cast pipe and 1.36 for centrifugal pipe cast in metal molds. This would indicate that the factors of safety given by the authors are somewhat too large.

It appears that the value of this paper would be increased if unit stresses and moduli of elasticity were given instead of loads and deflections. In dealing with specimens of different sizes, for which the needed dimensions are not all given, it is difficult to make an intelligent analysis of the results unless they are reduced to the intrinsic properties of the material itself.

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[1] General Manager, Pittsburgh Testing Laboratory, Pittsburgh, Pa.

[2] Professor of Engineering Mechanics, University of Michigan, Ann Arbor, Mich.

[3] Research Associate Professor of Theoretical and Applied Mechanics, Department of Theoretical and Applied Mechanics, College of Engineering, University of Illinois, Urbana, Ill.

[4] A. N. Talbot and F. E. Richart, "A Study of the Relation Between Properties of Cast-Iron Pipe Tested Under Impact, Internal Pressure, and Flexure, and the Corresponding Properties Found in Several Kinds of Test Specimens Taken Therefrom," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 185 (1926).

I should like to ask where the thickness measurements were taken, and how many; also on what part of the pipe the Brinell readings were taken? **Mr. Richart.**

Mr. Menefee. MR. MENEFEE.—In reference to the thickness of the pipe, pieces of pipe one foot long were taken from standard lengths. The minimum thickness on that one-foot length of pipe was then taken. We felt that we then had at least a reasonably close approximation to what would give us our tensile strength under bursting conditions. We felt that, all conditions being alike, and with no initial stresses present, the pipe would fail at the point of minimum thickness.

With reference to the Brinell tests, I do not believe I can answer that question; I think the readings were taken in several places. Naturally, our centrifugal pipe, chilled on the outside, would be a little harder than on the inside, and both kinds would be harder on the inside and outside surfaces than at a point intermediate. My memory does not serve me on that.

We felt that the formulas referred to, in which the efficiencies were calculated, were given as relative values rather than actual values, and I believe that rather careful reading will bring out a statement to that effect.

Mr. Richart. MR. RICHART.—The reason for my question regarding Brinell hardness and thickness was that our observations showed the thickness of the centrifugally-cast pipe to be exceedingly uniform around a circumference of the pipe, but to vary considerably along the length. As to the Brinell hardness, there is considerably more variation from inside to outside of centrifugally-cast pipe than for the ordinary pipe cast in a dry sand mold.

Mr. Moldenke. MR. RICHARD MOLDENKE.[1]—A question has been raised in respect to pipe of European manufacture. About a month ago I was in a French pipe foundry and saw some pipe ready for shipment to the United States, and while the pipe looked fine, yet when I saw some that had been tested and a split the entire length of the pipe, that made me wonder what we are going to find in this country after we have used French pipe a while, with our big trucks running over the ground and the serious shocks they are subject to. These pipe sometimes ran up to 1.8 per cent of phosphorus. I was therefore wondering whether, in these tests, pipe of such analysis had been considered in the comparisons.

It was very interesting to see some of the pipe for which steel rings had been put in the mold and the pipe cast into them, to stop this peculiar breaking.

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[1] Consulting Metallurgist, Watchung, N. J.

Mr. MacKenzie.

MR. J. T. MACKENZIE.[1]—I think it should be developed in the discussion here that in spite of the considerably higher strength of centrifugally-cast pipe, the paper shows higher load, especially in the compression test, on the sand-cast pipe, also the deflection shows higher on the centrifugally-cast pipe. In considering the known difference in the strength of the two materials it is evident that the sand-cast specimens tested were very much thicker than the centrifugally-cast pipe, because every test that has ever been made for modulus of rupture or tensile strength has shown certainly as much as 40 per cent, and in most cases up to 60 or 70 per cent greater strength, and they have also shown that the deflection on the equivalent sections is much lower in the metal mold centrifugally-cast pipe. As to that, in the paper by Talbot and Richart actual load-deflection curves are given in the case of pipe tested transversely as the whole pipe on 10-ft. supports, and it is found that the average for class 50 de Lavaud pipe is only 0.82 in. bending at the center at failure, whereas lot No. 1, which was a normal sand-cast pipe of very ordinary composition, such as would correspond closely with the analysis given in the present paper, shows 0.95 in. In that case the thickness of the sand-cast pipe was practically that reported by Menefee and White, or about 0.48 in. and the metal mold pipe was in the neighborhood of 0.30 in., so that in spite of the considerably greater thickness, the sand-cast pipe actually bent 15 per cent more.

Another point in the discussion is the oft-repeated statement that the annealing removes the strain from the pipe cast in the metal mold. I have never been able to bring myself to believe that. I think the statement that when a ring of the metal breaks it remains open—a known fact—is direct proof to the contrary. It also is disproved by the disparity between the tensile strength in internal pressure and the strength in cross-bending and impact. Now, consider that the inside layer of the material is actually pressing outward, of course the outside is being strained. In the impact test if you cut the outside—it is a very small layer to cut—the stress on the inside at once completes the rupture. It is the same way when you mount the pipe as a beam and load it transversely. The outside cracks first and then the whole pipe is easy to break. You will still have more strength than in sand-cast pipe in such a test, but the metal mold pipe does not develop the strength you would expect from a tension test. Now, when you load the pipe as in radial compression the tension is on the inside and the two elements of the pipe are working together as soon as the external pressure is equal to the internal strain. In the same way when you apply water pressure to the inside of the pipe you put on, say 10,000 lb. per sq. in. stress, which just equals the tension, and it takes the other 15,000 lb. stress to break the pipe. I use these particular figures for ease in illustration. I do not believe that the internal stress is that great, but that there is a considerable internal stress is to my mind beyond question. **Mr. MacKenzie.**

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[1] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.

MR. WALTER WOOD.[1]—There is one point in this paper of which I want to speak. There is a suggestion that the custom of the American cast-iron pipe manufacturers is to strip the pipe while at a high temperature. Some may do it, but it is not general. The custom in the American foundries is to strip the pipe only after they have lost all color, and while sometimes too hot for the hand, they yet have gotten practically cool, so that the annealing of sand-cast pipe is done in the mold and before it is stripped and the casting strains therefore practically eliminated. **Mr. Wood.**

MR. R. S. MACPHERRAN.[2]—Mr. Mackenzie raised a very interesting point about annealing. I can find no reference in the paper to annealing temperature. It is mentioned that much of the pipe is cast in a chilled mold. It must be annealed before it is put in service. At what temperature is it annealed and for how long? What tests have been made to indicate that all the casting strains are out and what affect did the annealing have on the physical properties? **Mr. MacPherran.**

MR. MENEFEE.—I might state that a little closer examination of the paper will bring out a paragraph to the effect that we found one pipe in the centrifugally-cast pipe which showed evidence of a cooling crack, the only piece we found; we thought we saw confirming evidence in the impact results, and that is mentioned in the paper, which at least led us to believe that the sand-cast pipe was freer from stress or strain due to cooling than the centrifugally-cast pipe. Those impact test results interpreted in that way, plus the fact that we found one piece of centrifugally-cast pipe with a crack not visible to the naked eye and which only came to light when we started to cut it, showing as it did that it was an old crack into which some of the surface coating had sunk a little way, led us to believe that this particular pipe, at least, had not had all of the cooling strains removed. There is some little comment in the paper on that. **Mr. Menefee.**

MR. A. N. TALBOT.[3]—I wish to refer to the type of flexure specimen used, given on page 26, called the transverse test specimen. As shown, in a flexure test the outer or convex side of the wall of **Mr. Talbot.**

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[1] Cast-Iron Pipe Manufacturer, R. D. Wood and Co., Philadelphia, Pa.

[2] Chief Chemist, Allis-Chalmers Manufacturing Co., West Allis, Wis.

[3] Professor Emeritus, University of Illinois, Urbana, Ill.

Mr. Talbot. the pipe will be in tension; if reversed, as was done in some of these tests, the inner part will be in tension. If the purpose of the test is to determine the modulus of rupture with the expectation that its value will be related to the tensile strength, and thus in some way be a measure of the bursting strength of the pipe, or if the deflection of the beam is to be used as a measure of the modulus of elasticity or the quality of the metal, as a flexure test may be used under proper conditions, the design of the specimen is open to criticism. The outside or inside of the wall may have the stronger or the weaker metal, flaws may be more likely to occur on one side of the metal or the other, all according to the method of manufacture used.

Since a purpose of such a test is to obtain a measure of the resistance of the wall to tensile forces throughout its whole thickness, the specimen should be designed so that metal throughout the thickness will be equally strained. A test specimen filling this requirement was described two years ago in the paper by Mr. Richart and myself, a form that proved very satisfactory in the large series of tests reported. In this test specimen the thickness of the wall constituted the width of the beam; the width of the strip formed the depth of the beam.

The so-called radial compression test of course is a flexure test. The difficulty with it is that four sections together form the beam resistance to be considered, two horizontal sections and two vertical sections, and that the division of resisting moment between them is indefinite and uncertain, although the sum of the resisting moments at a vertical section and a horizontal section remains constant. If the modulus of elasticity of the material were the same for all strains to the breaking point, as might be approximated in brittle metal, the resisting moment developed in one vertical section would be $0.091Qd$ and that in one horizontal section $0.159Qd$, Q being the applied load and d the diameter of the pipe. If the stress-strain diagram is markedly curved, as is found with a tough metal, the resisting moment developed in the vertical section will be less and that in the horizontal section greater than the values given above, both approaching in some degree to one half of the total resisting moment, that is to $\frac{1}{8}Qd$. As one purpose of the tests is to permit comparisons to be made of metal of different quality, the ring flexure test is not very satisfactory. Besides, as made, it would favor a pipe having higher tensile strength on the inside of the wall than on the outside.

Mr. Menefee. MR. MENEFEE.—With reference to Mr. Talbot's remarks concerning the transverse specimen, we recognized that that was open to some criticism. We wanted something to indicate, just as we said, what the relative resistances to bending were; we had it radially,

Mr. Menefee. so we wanted to get it longitudinally, and we adopted this method and presented the data as it was for such use as might be made of it. We gave it what we called a secondary status rather than primary. I have the impression that Mr. Richart thinks that our transverse tests favored the centrifugally-cast pipe. Our study of it led us to the opposite conclusion, and in the table on the last page we voted in favor of the sand-cast pipe so far as results on the transverse tests were concerned.

(*Author's closure by letter*).—Mr. Talbot's interpretation of the flexure test is not exactly correct. We obtained what we thought was a better representation of the tensile strength by direct tension tests. Our transverse tests were for the purpose of obtaining a qualitative idea as to how the two kinds of pipe could be expected to behave in service if subject to beam action, either of simple cantilever type, due to settling of trench or concentrated load from above.

The radial compression test is, as he says, a flexure test. We were not concerned with the formulas to which he refers, even though the modulus of elasticity were the same, which is not the case. We took two full-size sections and secured a quantitative comparison of the load resistance and corresponding deflection. We believe that the results can be interpreted directly into the service value of sand-cast and centrifugally-cast pipe of the same size. The refined points of unit resistances and division of resisting moments are interesting, but not pertinent, since we were not comparing "metal of different quality" so much as we were comparing pipe of different quality. Pipe encompasses all the differences: dimensions, heat treatment, quality of metal, and any other property that would influence its service life.

GAS WELDING OF HIGH-CHROMIUM CORROSION-RESISTANT ALLOYS

BY W. B. MILLER[1]

SYNOPSIS

With the increasing use of high-chromium alloys to withstand corrosion and oxidation at high temperatures, there has developed a need for information concerning the welding of these alloys. The paper gives information on the welding properties of six groups of chromium alloys in wrought form, comprising stainless steel, three groups of stainless iron, high-chromium high-nickel alloys and high-chromium alloys. The welding of high-chromium castings is also discussed. The tensile and bend test properties of the three groups of stainless iron in the form of welded sheets are given, which show the value of annealing the sheets after welding. The importance of using a suitable flux in the welding of these alloys is emphasized.

INTRODUCTION

The use of high-chromium alloys as engineering materials to withstand corrosion and oxidation at high temperatures is increasing rapidly. As may be expected, there is an insistent demand for information concerning the welding of these alloys, since they present difficulties not usually encountered in the welding of ordinary low-carbon steel. Their utilization will increase with the possibility of welding the steels. As with all other metals, success in welding high-chromium alloys can be attained only through a thorough knowledge of the properties of the various alloys when subjected to high temperatures, such as are met with in welding practice. Attempts have been made to weld the various chromium alloys by those not having a clear understanding of their properties, with the result that the alloys have been unjustly criticized.

There are now on the market no less than fifty-five chromium alloys, the products of several manufacturers, the same alloy often being known under a variety of trade names. No attempt will be made to classify the alloys according to their trade names, as such a classification would require needless repetition and would be confusing. Practically all the commercial alloys can be divided for the purposes of this paper into seven groups according to chemical composition. The welding data for each group will be given, the procedure for a given group being the same regardless of the variations in chemical composition which may occur within it.

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[1] Union Carbide and Carbon Research Laboratories, Inc., Long Island City, N. Y.

GROUP I.—STAINLESS STEEL

Group I, Stainless Steel, is defined by the following composition range:

Chromium	12.0–16.0 per cent
Carbon	over 0.20 "
Other elements in amounts incidental to method of manufacture	

A typical analysis is as follows:

Carbon	0.33 per cent
Manganese	0.83 "
Silicon	0.35 "
Chromium	12.75 "

Stainless steel of the above type is used for cutlery and heat-treated parts having general corrosion resistance and resistance to wear and abrasion. Stainless steel is very susceptible to heat treatment, its corrosion resistance depending on its heat treatment and surface finish. On account of the air-hardening properties of these alloys, the welds and metal adjacent to the welds will be hard and brittle. This brittle condition may be remedied somewhat by annealing followed by slow cooling. After this treatment the metal will not have its maximum stainless properties, which are developed only by quenching-and-drawing operations followed by proper surface finishing. On account of the hardness and brittleness of welds in stainless steel, and the difficulty of properly heat treating and surface finishing the material after welding, stainless steel is unsuitable for welded parts subjected to corrosion coupled with severe stress or vibration, unless the stress is carried by other construction.

GROUP II.—STAINLESS IRON

Group II, Stainless Iron, is defined by the following composition range:

Chromium	12.0–15.0 per cent
Carbon	less than 0.12 "
Other elements in amounts incidental to method of manufacture	

A typical analysis is as follows:

Carbon	0.10 per cent
Manganese	0.33 "
Silicon	0.39 "
Chromium	13.20 "

The difference between stainless steel and stainless iron lies chiefly in the carbon content, and heat treatment is not required to develop the corrosion-resistance properties of this low-carbon alloy. Stainless iron has good corrosion resistance to many agents which readily attack iron, steel and non-ferrous alloys. This material is generally furnished in sheet form, flange welds being usually employed for thicknesses up to No. 16 gage (0.0625 in.). In order to weld sheet material the flange, preferably about $\frac{1}{16}$ in. high, is painted top and bottom with a water paste of special flux after which the weld is accomplished by fusing together the flanges. A neutral flame just large enough to insure proper fusion should be used, since too much heat causes the molten metal to boil, resulting in porous welds. A neutral flame should be maintained during the welding operation, since excess acetylene will cause an increase in the carbon content of the weld and lower its resistance to corrosion. It will also increase the hardness and lower the ductility of the weld metal. An excess of oxygen causes the formation of an excessive amount of infusible oxide resulting in porous welds.

With the aid of a proper flux and rods of the same composition as the base metal, good welds can be produced in metal $\frac{1}{16}$ to $\frac{1}{8}$ in. thick without beveling. It is essential that the flux be applied to the line of the weld on both the top and bottom surfaces of the sheet. This prevents oxidation on the underside, allowing good fusion and union to take place under the protecting film of slag produced. Without the use of a proper flux it will be impossible to obtain complete fusion at the bottom of the seam. In making welds in heavier gages the plate is beveled and the faces of the bevel are coated with the flux. The welding should be carried through to completion as rapidly as possible and should be performed on one side only. Welding on both sides can be accomplished with the aid of a high preheat, but this practice is not recommended in ordinary cases.

Stainless iron is self-hardening or air-hardening when heated above its critical temperature, 790 to 800° C. Welds and metal adjacent to the weld in stainless iron will consequently be hard and brittle. Proper subsequent heat treatment will remedy the brittleness to a large extent. Good results are obtained by heating to 650 to 750° C. for one-half hour and cooling either in the furnace or in still air. The welding torch may be utilized for annealing where it is not possible to use furnaces, though furnace annealing is recommended. When using the torch for annealing, especial care must be exercised to avoid heating the material to 790° C. or above. The benefit to be derived from torch annealing will depend on the temperature and the time of heating. Usually about five minutes within the above temperature range will appreciably soften the metal affected by the welding heat.

Table I shows the properties that may be expected in welds in No. 18 gage (0.05 in.) stainless iron of the above composition. Before welding, the sheet was annealed at 750° C. and air cooled. The tensile properties of the sheet and welds before and after annealing were determined on coupons $1\frac{1}{2}$ in. wide. A welded test specimen is nonhomogeneous, the weld having a higher yield point than the base metal. When such a test specimen is broken in tension, the base metal adjacent to the weld is not free to elongate due to the propping or staying action of the stiffer weld metal. Consequently the percentage of elongation obtained in the tension test of a welded coupon usually does not express the true ductility of the weld metal or metal adjacent to the weld. When the weld has a higher yield point than the base metal, no appreciable lateral contraction takes place at the weld. Bend tests were made in which the specimen was forced to bend at the weld by squeezing in a vise. The method which has been described in other papers[1] has proved to be the most satisfactory means of securing information about the ductility of welds. This is different from the American Welding Society method of bend testing where the specimen is bent around a pin. Both the percentage of elongation in 2 in. over the weld and the angle of bend have been recorded, but the greater emphasis should be placed on the bend angle results. The bend tests were made transversely to the line of the weld, which gave an indication of the ductility of the weld and adjacent metal. On account of the thinness of the sheet and the small radius of curvature, it was impossible to measure the percentage of elongation of the outside fibers in the bend test. The angle of bend was therefore measured. This method does not give quantitative data, but gives information about the ductility of the weld.

TABLE I.—TENSION AND BEND TESTS, OF STAINLESS IRON, 12 TO 15 PER CENT CHROMIUM.

Sheet 0.05 in. thick. Average values.

	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in. over Weld, per cent	Bend Angle, deg.
Sheet as received	53 600	78 600	34.0	180
Sheet annealed	48 800	77 200	34.0	180
As welded	32 400	43 700	4.0	33 (Brittle)
Torch annealed	55 350	76 400	6.5	85 (Medium ductility)
Furnace annealed	45 000	70 800	7.0	140 (Excellent ductility)

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[1] W. B. Miller, "The Bend Test as Applied to Welded Coupons," *Journal*, Am. Welding Soc., September, 1927; A. B. Kinzel, "A Critical Study of the Bend Test as Applied to Iron and Steel," *Transactions*, Am. Soc. Steel Treating, November, 1927.

In the as-welded condition, welds in stainless iron are quite brittle and this fact should be considered in applying welding to this material. Welds should always be heat-treated to restore toughness and ductility to the metal affected by the heat.

GROUP III.—STAINLESS IRON

Group III, Stainless Iron, is defined by the following composition ranges:

Chromium	16.0–20.0 per cent
Silicon	0.5– 1.5 "
Carbon	less than 0.12 "

A typical analysis is as follows:

Carbon	0.08 per cent
Manganese	0.29 "
Silicon	1.05 "
Chromium	17.21 "

The directions for welding rustless irons with the lower chromium contents apply also to this grade. The higher silicon content in the 16 to 20-per-cent chromium grade facilitates the welding, alloys containing about 1 per cent being more easily welded than those containing less than 0.5 per cent. The stainless irons containing 16 to 20 per cent of chromium and low carbon do not air-harden so readily as the 12 to 14-per-cent chromium alloys. They are, however, liable to grain growth at welding temperatures and possibly for this reason welds and metal adjacent to the weld in the higher chromium stainless irons are considerably reduced in ductility. Torch and furnace annealing at temperatures of 650 to 750° C., followed by air cooling, will impart toughness and ductility to welds in this alloy. The tension and bend test values in Table II show the strength and ductility of welds in No. 18 gage sheet.

TABLE II.—TENSION AND BEND TESTS OF STAINLESS IRON, 16 TO 20 PER CENT CHROMIUM.

Sheet 0.05 in. thick. Average values.

	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in. over Weld, per cent	Bend Angle, deg.
Sheet as received	63 500	77 100	31.0	180
Sheet annealed	46 700	75 000	29.0	180
As welded	41 500	53 400	1.0	25 (Brittle)
Torch annealed	47 600	68 900	6.0	110 (Good ductility)
Furnace annealed	50 800	73 800	11.6	150 (Excellent ductility)

Furnace annealing has improved both the strength and ductility of welds in this type of alloy. Oxy-acetylene welded tubing, after a low temperature furnace anneal, may be flattened and crushed showing no cracks in the weld or metal adjacent to the weld. Such tests also show the thorough fusion obtained at the bottom of the seam with the aid of a flux. Annealed welded tubing may also be cold-drawn considerably, indicating excellent ductility.

GROUP IV.—STAINLESS IRON

Group IV, Stainless Iron, is defined by the following composition ranges:

Chromium	17.0–25.0	per cent
Nickel	7.0–12.0	"

A typical analysis is as follows:

Carbon	0.12	per cent
Manganese	0.32	"
Silicon	0.13	"
Chromium	17.42	"
Nickel	7.58	"

Stainless irons of this type are austenitic and are not subject to the brittleness encountered in the previously mentioned types of alloys as a result of the high welding heat. They do not air-harden and weld metal cooled either slowly or rapidly will not be hard and will possess considerable ductility. These alloys are particularly suitable for welded construction since they require no subsequent heat treatment. The metal can be welded without the use of a flux, but superior results are obtained through its use, for only with a suitable flux can good bottom fusion be obtained. In Table III are shown the results of tension and bend tests on No. 18 gage sheet of the above composition.

TABLE III.—TENSION AND BEND TESTS OF STAINLESS IRON, 17 TO 25 PER CENT CHROMIUM.

Sheet 0.05 in. thick. Average values.

	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in. over Weld, per cent	Bend Angle, deg.
Sheet as received	53 500	102 100	68.0	180
Sheet annealed	58 800	104 300	68.0	180
As welded	54 200	66 200	15.0	150
Torch annealed	50 400	58 000	9.0	140
Furnace annealed	56 500	74 000	24.5	180

Since welds in this alloy show the best combination of strength and ductility in either the as-welded condition or furnace-annealed condition, it is considered the best high-chromium alloy to use for welded construction. Single V welds in material $\frac{1}{8}$ in. thick have been made showing a tensile strength of 80,000 lb. per sq. in. with 32 per cent elongation in 2 in. over the weld. The properties to be obtained in welds in this type alloy are dependent to a considerable degree on the quality and physical properties of the metal before welding. For instance, moderately cold-worked light-gage sheet usually yields higher weld strengths than fully annealed material. Water quenching from high temperature improves greatly the cold-working properties of welds in this type of alloy. On account of distortion this drastic treatment can be applied to only a few welded products. Oxy-acetylene welded tubing may be cold-drawn to size without annealing.

GROUP V.—HIGH-CHROMIUM HIGH-NICKEL ALLOYS

Group V, High-Chromium High-Nickel Alloys, is defined by the following composition ranges:

Chromium	7.0–25.0	per cent
Nickel	17.0–22.0	"
Silicon	1.0– 3.0	"

The alloys in this group have been developed to meet certain special needs in industry. They are easily welded, although in heavier gages care should be exercised to control expansion and contraction, since some of these alloys are quite liable to be red short and crack in the weld just after solidification. Welds in these alloys do not require subsequent heat treatment for they have properties similar to the alloys in Group IV.

GROUP VI.—HIGH-CHROMIUM ALLOYS

Group VI, High-Chromium Alloys, is defined by the following composition ranges:

Chromium	20.0–30.0	per cent
Carbon	less than 0.3	"

These alloys are characterized by great resistance to chemical corrosion and to oxidation at high temperatures. On account of their high chromium content they present some difficulty in welding, particularly on heavy gages, due to the formation of an infusible oxide or slag. With the use of the proper flux, as previously outlined, very satisfactory flange and butt welds can be made in this alloy. This composition is not self-hardening, but does experience grain growth at welding temperatures. Welds and metal adjacent to the weld will have strength but very little ductility. This brittleness of weld and adjacent metal is not necessarily an obstacle when it is considered that one of the greatest fields for this alloy is to resist oxidation at elevated temperatures. Welds and adjacent metal in 20 to 30-per-cent chromium alloys are ductile at a good black heat; consequently this brittleness is considerably overcome by and during operating temperatures. Many instances can be cited of welded apparatus performing satisfactorily at elevated temperatures. Due to its high chromium content this metal is particularly suited for resisting severe corrosion conditions. It resists oxidation at temperatures above 1050° C. and is resistant to sulfur fumes at high temperature.

GROUP VII.—CASTINGS

There are on the market several grades of high-chromium castings, some containing appreciable amounts of nickel or silicon or both. Castings, as a rule, contain considerably more carbon than the forged products. They are used for resistance to corrosion, heat or abrasion, the composition being determined by the use.

In welding castings it is preferable to preheat to a good red heat to avoid cracking. Cooling should be relatively slow, after which the castings may require further heat treatment to produce the necessary hardness. Welding rods of the same composition as the base metal should be used. An excess acetylene flame aids in welding and is allowable for castings which already have a high carbon content.

Tests have shown that for all the various high-chromium alloys, a suitable flux is required for best results in oxy-acetylene welding. The importance of the flux cannot be emphasized too strongly, for it allows the use of a neutral flame and blankets the fusion zone, preventing excess atmospheric oxidation or carburization by the flame. Without it good fusion cannot be obtained at the bottom of the seam. A suitable flux is one which will readily dissolve chromium and iron oxides and produce a slag which is fusible at the welding temperatures of these alloys and which is neither too viscous nor too fluid, so that the weld will be well protected from the surrounding atmosphere and so that any impurities will quickly float to the surface of the weld. The ordinary fluxes used for general welding purposes will not accomplish these results, which are necessary if good welds are to be secured. All surfaces to be welded should be clean and free from oxide or mill scale. Before welding any particular alloy, one must consider carefully the properties of the metal, such as the effect of the welding heat, and heat treatment, if required, to produce the necessary toughness and ductility in the weld. A knowledge of the proper heat treatment for the various alloys is essential if the maximum combination of strength and ductility is desired.

DISCUSSION

Mr. Gillett.

MR. H. W. GILLETT.[1]—Of what is this special flux made? The author refers in the paper to a "suitable and proper" flux, without even a statement as to the general chemical nature of that flux. I do not think we ask for proprietary details, but I do not feel that it is "suitable and proper" for a paper of this sort to be presented to the Society without more information on so essential a point. No one could go ahead with any application whatever of the other information in the paper without knowing what flux to use. For all practical purposes the paper, without information on the flux, is worthless. If commercial considerations will not yet allow giving facts about the flux, the paper is not yet ripe for publication. I am surprised that a paper from a laboratory of such high standing as this one should lack data on so important a link in the chain of argument, and equally surprised that the Papers and Publications Committee should accept a paper with such a vital part left out. I think that before publication that vital part should be put in.

Mr. Hall.

MR. JOHN HOWE HALL.[2]—The author makes quite a point of the fact that the ductility of these welds should not be measured by the percentage of elongation over the weld, but should be measured by the bend. Glancing over the figures, it strikes me that the reason he holds that opinion is that the elongation is poor and the bend is good. I suppose I ask simply from ignorance, but I cannot understand why we should not pay any attention to the elongation and should judge the efficiency of the welding chiefly by the bend, especially if my understanding is right that the bends are made perpendicular to the line of the welds.

Mr. Moore.

THE PRESIDENT (*Mr. H. F. Moore*[3]).—I wonder if the Chair may be allowed a very brief word in addition to what has been said? He would like to refer to the work which the bend section of Committee E-1 on Methods of Testing is doing, measuring elongation in a bend specimen. A method is being developed by the section under the leadership of Mr. Kinzel, and he has already described that method in a paper before the American Society for Steel Treating.[4]

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[1] Chief, Division of Metallurgy, U. S. Bureau of Standards, Washington, D. C.

[2] Metallurgical Engineer, Taylor-Wharton Iron and Steel Co., High Bridge, N. J.

[3] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[4] A. B. Kinzel, "A Critical Study of the Bend Test," *Transactions*, Am. Soc. Steel Treating, November, 1927.

Mr. Moore. It is a promising method involving a combination of elongation with the bend testing, though the method is by no means completely developed as yet.

Mr. Lessells. MR. J. M. LESSELLS.[1]—With regard to this question of measuring elongation, we are conducting at present investigations on the strength of arc-welded joints where we are measuring elongation directly, using special extensometers which have small gage length. The elongations we get, measured directly, give some very interesting information. I believe that elongation should be measured directly, as in the tension test, and not by the bend test, because I think in this latter test there is a factor that cannot be controlled. If a welded joint is subjected to tensile stresses, such as centrifugal stresses, we want to know what the deformations and elongations are, so I should strongly recommend that any data on the welded joints be determined directly as in the tension test.

Mr. French. MR. H. J. FRENCH.[2]—I should like to ask the author to be a little more specific about the compositions of the castings to which he refers.

Mr. Smith. MR. H. E. SMITH.[3]—There are two features involved in testing a weld: one is the integrity of the weld metal and the workmanship of the welding, and the other is the integrity of the structure as a whole; and that is where the difference in elongation comes in. If, for example, we weld a firebox of mild steel (carbon 0.12 to 0.15 per cent) with copper, as used to be done, we have only the strength of copper, even if the weld is perfect. If we make a tension test, before the steel can elongate over the gage length of 8 or 2 in. the copper is broken, so obviously we do not get much elongation. In judging welding we must consider, first, whether the weld is perfect, and then whether, with a perfect weld, our structure is satisfactory and safe. We have made a good many tests of the sort mentioned and of course always get a low elongation when welding with a material that is weaker than the material welded.

Mr. Miller. MR. W. B. MILLER[4] (*author's closure by letter*).—Mr. Gillett has requested the composition of the flux recommended. The composition of the flux used was not incorporated in the paper for the reason that it is a patented composition and we did not wish to inject anything of an advertising nature into the paper. Several fluxes may be compounded which will work better than the usual welding fluxes on

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] Engineer of Materials, New York Central Lines, New York City.

[4] Union Carbide and Carbon Research Laboratories, Inc., Long Island City, N. Y.

Mr. Miller. the market. We have found a fused mixture of borax and silica, made by fusing from two to four and one-half parts of ordinary borax with one part of silica, to be the most satisfactory. Such a composition has a high solvent power for metallic oxides, is highly resistant to heat and protects the fusion zone and adjacent hot metal from oxidation. It has the further advantage of being practically water insoluble, and therefore may be applied as a water paste without intumescing and blowing away when heated.

The question has been raised as to the value of the bend test for evaluating welds. A welded coupon is non-homogeneous in respect to physical properties. Welds usually have a higher yield point than the base metal. Whenever fracture in a tension test specimen occurs outside the weld, the percentage of elongation, usually measured in 2 in. over the weld, is not a true measure of the ductility of the weld metal. With gage lengths including only weld metal, the bend test gives a true measure of the ductility of the weld metal. For more detailed information on the bend test the reader is referred to the paper by A. B. Kinzel, mentioned by Mr. H. F. Moore[1] and a paper by the author presented before the American Welding Society.[2]

Regarding Mr. French's inquiry as to the composition of castings mentioned in the paper, we refer to heat and corrosion-resisting castings containing 18 to 28 per cent of chromium with carbon 0.5 to 3.00 per cent; also to castings containing 8 to 28 per cent of chromium, 6 to 40 per cent of nickel, 1 to 3 per cent of silicon, usually with low carbon.

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[1] *Loc. cit.*

[2] W. B. Miller, "The Bend Test as Applied to Welded Coupons," *Journal*, Am. Welding Soc., September, 1927.

THE SEIZING OF METALS AT HIGH TEMPERATURES

By N. L. Mochel[1]

Synopsis

This paper describes apparatus which has been devised in which a specimen of any material is caused to rotate in contact with a stationary specimen of any material, under various loads and at any temperature, to determine whether or not those materials will seize or bind when operating in contact with each other without lubrication. Results of preliminary tests at 750° F. on a number of materials are given. Materials which by experience are known to seize in service, quickly seized in the test apparatus. Other materials behaved in such manner as to indicate they are immune to this action of seizing.

Introduction

In the construction of valves and other equipment designed to operate under high-temperature steam conditions, it is often necessary to have metal parts working in contact with each other without lubrication. A troublesome feature in the operation of such equipment is the sticking or seizing of the parts because of the tendency of many materials to so seize or gall when under such service conditions. The action is by no means limited to high temperatures but may occur with some materials at normal temperatures. In general, however, it is believed that high-temperature conditions increase the tendency towards seizing.

In the choice of materials for the parts of such equipment, a knowledge of how the materials under consideration will operate when in working contact with each other is very necessary. In an endeavor to provide such information, test apparatus was devised in which specimens could be made to rotate in contact with each other, under any desired speed, load and temperature. This paper describes the test apparatus and gives the results of the preliminary tests on a number of materials.

Test Apparatus

The arrangement of the test apparatus is shown in Fig. 1. Test specimens and holders, 1 and 2, are further shown in detail in Fig. 2. The lower specimen remains stationary and rests in a spherical seat to insure alignment. The upper specimen is secured in the tapered shank holder, which fits the revolving spindle 33. Worm 31 is direct-

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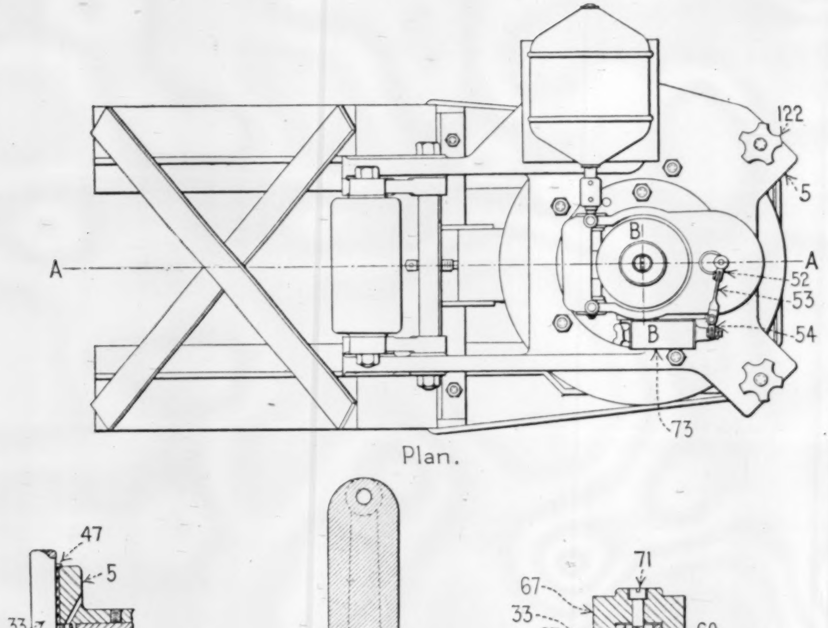
[1] Metallurgical Engineer, Westinghouse Electric and Manufacturing Co., Philadelphia, Pa.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5
15
2
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72
118
114
116

Plan.

Section B-B, Showing Thermocouple Well.

Vertical Section A-A.

FIG. 1.—Test Apparatus for Determining Seizing of Metals at High Temperatures.

connected to the motor, rotating worm wheel 32, which drives sleeve 35 through four pins. Sleeve 35 transmits rotation to the revolving spindle 33, carrying the specimen, by means of a piece of 0.020 in. steel wire held in hardened sleeves, as at 60. Thus the weight of the spindle 33, and additional weights placed on it such as 67, are directly on the specimens, with of course some loss by reason of friction through the bearing and the effect of the 0.020-in. wire. When seizing of the specimens takes place, the driving wire shears or is pulled out of the holder. The revolving spindle and the rotating specimen can readily

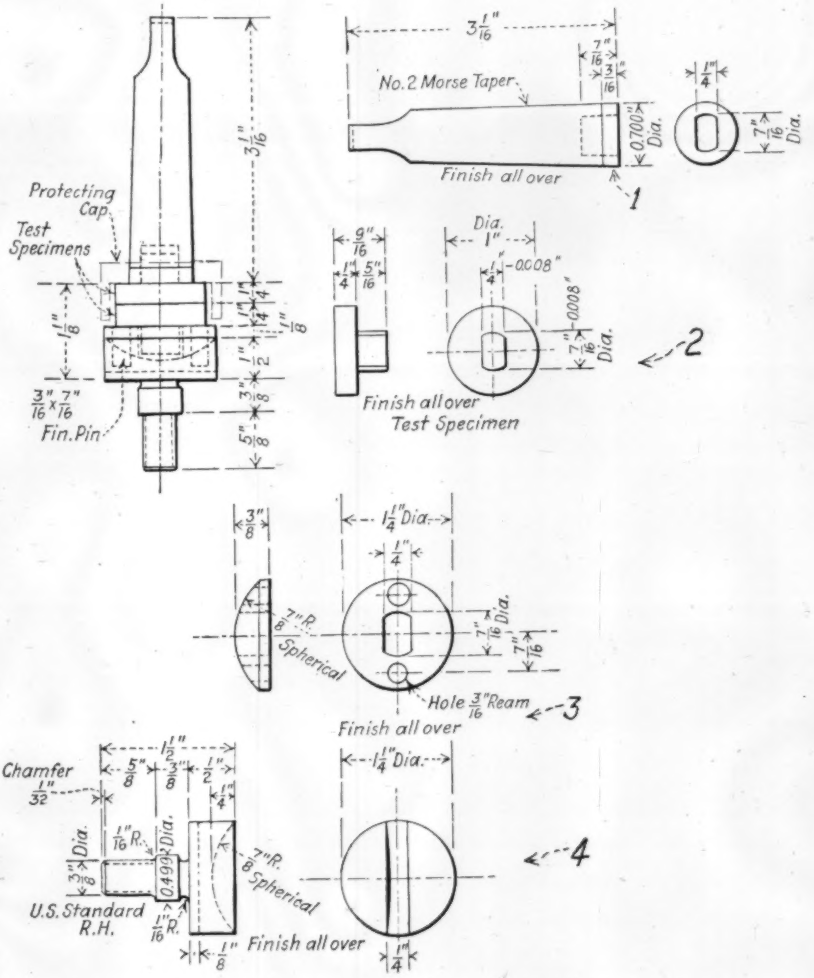

Protecting Cap
Test Specimens
3/16" x 7/16" Fin. Pin
3 1/16"
No. 2 Morse Taper
Finish all over
0.700" Dia.
7/16" Dia.
1/4"
1
Dia. 1"
1/4" -0.008"
-0.008"
9/16"
5/16"
Finish all over
Test Specimen
2
1 1/4" Dia.
7/8" R. Spherical
Hole 3/16" Ream
Finish all over
3
Chamfer 1/32"
1 1/2"
5/8"
3/8"
1/2"
1/16" R.
0.499" Dia.
3/8" Dia.
U.S. Standard R.H.
7/8" R. Spherical
Finish all over
4

FIG. 2.—Details of Holders and Test Specimens.

be removed by hand without dismantling the other parts. Gears 41 and 42 drive a revolution counter at 48 and 73. The casing in the furnace which carries the lower specimen, the spindle and the specimen holders are made of non-scaling materials.

The electric furnace is controlled through resistance and an automatic control pyrometer to ±10° F. of the desired temperature. The thermocouple well 63 was later removed and the bare metal couple inserted directly against the stationary specimen.

A selection of motors, worms and worm wheels varies the speed of revolution.

Test Specimens:

The same type specimen was used for both top and bottom, as shown in Fig. 2. The contact surface of all specimens was finished on "00" Manning emery paper, being held in a simple grinding fixture during this operation, which insured flatness of the contact surface, and further insured the contact surface being closely at right angles to the axis of the specimen.

Test Procedure.—All tests reported in this paper were carried out at 750° F. After the specimens were properly secured in position and the machine clamped down, the temperature was slowly raised until the thermocouple registered a specimen temperature of 750° F. The motor was then started, causing the top specimen to rotate on the lower one. For the first series of tests, an actual load of 20 lb. and a speed of rotation of 235 r. p. m. were used. When seizing occurred, the driving wire was sheared or pulled out, and the test discontinued.

Later, tests were made on some materials up to 100 hours duration, and with higher loads.

Materials Tested

The materials tested are shown in the following table:

Designation	Material
CI-1	Cast Iron: Si 1.10 per cent, Total Carbon 3.40 per cent
CI-2	Cast Iron: Si 1.43 per cent, Total Carbon 3.33 per cent, Ni 2.70 per cent, Cr 0.50 per cent
CI-3	Cast Iron: Si 0.45 per cent, Total Carbon 3.82 per cent, Ni 3.97 per cent
CI-4	Cast Iron: Si 0.48 per cent, Total Carbon 2.60 per cent, Cr 1.48 per cent, Monel Metal 20 per cent
CI-5	Cast Iron: Si 0.70 per cent, Total Carbon 2.96 per cent, Ni 3.47 per cent
CI-6	Cast Iron: Si 1.39 per cent, Total Carbon 2.68 per cent, Ni 2.14 per cent
CrPl-1	Carbon Steel (0.40 per cent carbon), chromium plated by one laboratory
CrPl-2	Carbon Steel (0.40 per cent carbon), chromium plated by second laboratory
NIT	Nitrided Steel
MM	Rolled Monel Metal

DESIGNATION	MATERIAL
SI-1	Stainless Iron: Cr 12.40 per cent, C 0.12 per cent, quenched and drawn, Brinell hardness 223
SI-2	Stainless Iron: Cr 12.40 per cent, C 0.12 per cent, annealed, Brinell hardness 180
NiBr	Cast Nickel Bronze: Cu 84 per cent, Sn 10 per cent, Zn 1 per cent, Ni 5 per cent
Stl	Rolled Carbon Steel, S. A. E. No. 1040
NiCr	Cast Nickel Chromium: Steel Ni 20 per cent, Cr 7 per cent
Ni-1	Rolled Nickel: C 0.09 per cent, Si 0.02 per cent, Fe 0.54 per cent, Cu 0.22 per cent, Ni remainder
Ni-2	Rolled Nickel: C 0.16 per cent, Si 0.45 per cent, Fe 0.70 per cent, Cu 0.19 per cent, Ni remainder
Ni-3	Rolled Nickel: C 0.28 per cent, Si nil, Fe 0.18 per cent, Cu 0.22 per cent, Ni remainder
Ni-4	Rolled Nickel: C 0.29 per cent, Si 0.53 per cent, Fe 0.46 per cent, Cu 0.23 per cent, Ni remainder
CuNi	Cast Copper-Nickel Alloy: Cu 63 per cent, Ni 30 per cent, Fe 7 per cent
WS	Cast Tungsten Steel: C 0.25 per cent, Cr 1.94 per cent, W 13-0.0 per cent
CrS-1	Cast Chromium Steel: C 0.65 per cent, Cr 17.95 per cent, Ni 0.42 per cent
CrS-2	Cast Chromium Steel: C 0.81 per cent, Cr 15.88 per cent, Ni 0.46 per cent
CrS-3	Cast Chromium Steel: C 1.02 per cent, Cr 19.13 per cent, Ni 0.42 per cent
CS	Cast Steel: C 0.30 per cent

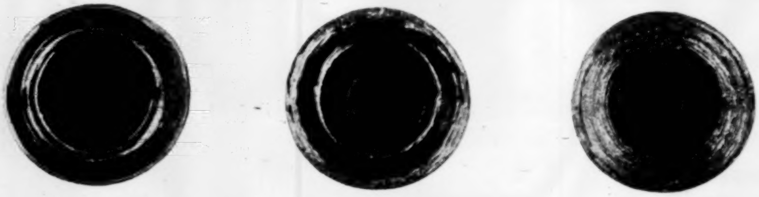
TEST RESULTS

Comparison of results will be made on the basis of the rotating or upper specimens.

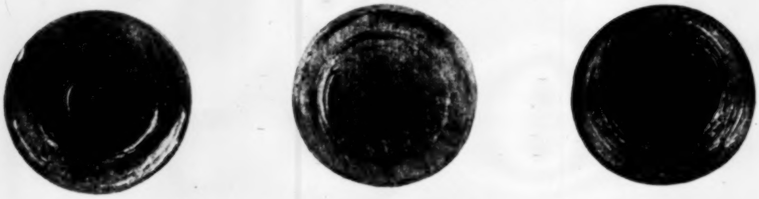
Monel Metal (*MM*) seized very quickly when operating on itself, after but 35 revolutions. It also seized after 3515 revolutions on nickel bronze (*Ni Br*), at once on stainless iron (*SI-1*), after 45 revolutions on copper-nickel alloy (*Cu Ni*), and at once on nickel (*Ni-1*). It operated very satisfactorily on all six cast irons, on the cast steel (*CS*), on chromium plating (*CrPl-1*), on the nitrided steel (*NIT*), on the cast tungsten steel (*WS*), and on all three cast chromium steels (*CrS-1, 2, 3*). Especially did it operate well on the nitrided steel, one test at 20-lb. load being run for 100 hours and another at 40-lb. load being carried for 35 hours without seizing.

Stainless Iron (*SI-1*) seized very quickly when operating on itself, after 205 revolutions. It also seized after 1674 revolutions on nickel bronze (*NiBr*), after 2357 revolutions on nickel (*Ni-2*), and after 113 revolutions on the copper-nickel alloy (*CuNi*). It seized at once on monel metal (*MM*), cast nickel-chromium steel (*NiCr*) and on the other three nickels. It operated very satisfactorily on the nitrided steel (*NIT*), on all six cast irons, on the cast steel (*CS*), on the chromium plating (*CrPl-1*), on the cast tungsten steel (*WS*), and on all three cast chromium steels (*CrS-1, 2, 3*). An interesting item is the ability of stainless iron (*SI-1*) to operate on stainless iron (*SI-2*) without seizure. One such test was carried for 4 hours duration. Excessive wear takes place with this combination, and it is of further interest to note that the harder material wears more than the softer. In one of the tests of *SI-1* on *CrS-1*, the pyrometer control motor failed, and the temperature was finally determined as 1400° F. The two materials had continued to operate satisfactorily as the temperature increased.

Nickel of all four types seized at once with any combination of the four. It also seized at once with monel metal (*MM*) or stainless iron (*SI-1*). Nickel (*Ni-1*) operated without seizure on the cast tungsten steel (*WS*) and on all three cast chromium steels (*CrS-1, 2, 3*).



Stainless Iron (SI-1) Monel Metal (MM) Stainless Iron (SI-1)



Monel Metal (MM) Monel Metal (MM) Stainless Iron (SI-1)

FIG. 3.—Illustrating Specimens After Seizing Test. The specimens in the upper row were rotated respectively on the specimens in the lower row.

Steel (*Stl*) seized at once when operating on itself. But it operated very well on the three cast chromium steels (*CrS-1, 2, 3*).

Nitrided Steel (*NIT*) operated well against itself, nickel bronze (*NiBr*), chromium plating (*CrPl-1*), monel metal (*MM*) and stainless iron (*SI-1*).

Chromium-Plated Steel (*CrPl-1, 2*) operated well with themselves, and with monel metal (*MM*) and stainless iron (*SI-1*). It was observed that the plating flaked off (*CrPl-2*) during test.

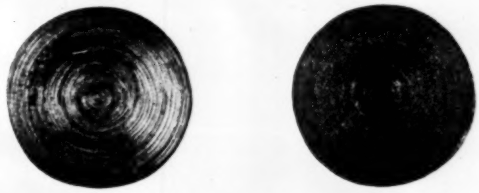
Nickel Bronze (*NiBr*) operated on itself without seizing, but seized on monel metal (*MM*) and stainless iron (*SI-1*).

Cast Iron (*CI-1*) operated well when in contact with all the cast irons.

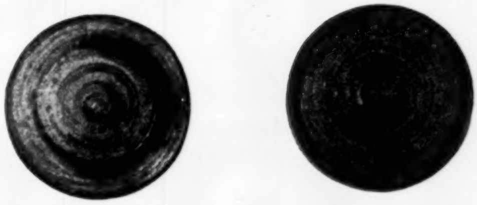
Figures 3 and 4 show the appearance of a few specimens after test. In Fig. 3 are shown combinations of *SI-1* on *MM*, *MM* on *MM*, and *SI-1* on *SI-1*, which quickly seized under test. In Fig. 4 at the left are shown the specimens of *SI-1* on *SI-2* after four hours test, and at the right *MM* on *NIT* after 100 hours test.

CONCLUSIONS

From field experience, it was expected that certain of the above combinations would seize under test. The tendency of copper-nickel alloys to behave in this way is quite well known. Likewise, it was expected that other materials could be operated against cast iron, for example, without trouble. It was believed that if the test apparatus would confirm these field experiences, it could be used in predicting the behavior of other materials or newly developed ones when in such service. The results do confirm in many respects the experiences encountered in service.



Stainless Iron (SI-1) Monel Metal (MM)



Stainless Iron (SI-2) Nitrided Steel (NIT)

FIG. 4.—Illustrating Specimens After Seizing Test. The specimens in the upper row were rotated respectively on the specimens in the lower row.

From these preliminary tests, the conclusion is reached that a test method of practical value has been developed. Nothing more is claimed, as considerable work now in progress must be completed before further discussion and conclusions are offered.

Acknowledgments.—The author wishes to acknowledge the assistance of Mr. E. A. Snader in carrying out the tests, and the cooperation and helpful suggestions of Mr. R. Marsland, Turbine Engineer.

DISCUSSION

Mr. French.

MR. H. J. FRENCH.[1]—In the described equipment there are two approximately plane surfaces in contact. The extent of contact will be dependent upon the smoothness and accuracy of the surfaces and I should like to ask whether this has been taken into account. In other words, can the results be said to be representative of the materials under test or are they more nearly indicative of the effectiveness of the finishing operations in the preparation of the test specimens? A few comments on this subject would help in interpreting the data presented.

Mr. Mochel.

MR. N. L. MOCHEL.[2]—The question raised by Mr. French is an important one. Considerable thought was given to this matter. The preparation of the test specimens is covered briefly in the paper. All specimens were finished to the same smoothness and accuracy, that is, flatness of contact surface and contact surface at right angles to the axis. During the development of the test procedure, the spherical seat for the lower specimen was so constructed as to cause leaning over in the direction of rotation and partial contact only of the upper specimen on the lower was secured. But the results secured agreed in all ways with later results when the spherical seat was corrected to give full contact. Further, other tests were made in which the lower specimen remained as shown in the paper, but the upper specimen was machined as shown in the accompanying Fig. 1. These tests gave results in no way conflicting with those given in the paper.

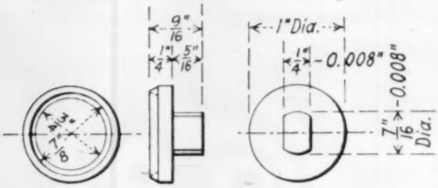


FIG. 1.—Details of New Test Specimens.

The author is reminded at this time of a comment made by Mr. Gillett during the symposium on wear testing, to the effect that

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[1] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[2] Metallurgical Engineer, Westinghouse Electric and Manufacturing Co., Philadelphia, Pa.

Mr. Mochel. in developing test equipment and procedure for wear testing, one must first go out and get the results, that is, one must make that procedure give results agreeing with known service results, and only then can it be used as a test method. As stated in the conclusions offered in the paper, the author assumed a like position, that is, that if the test method confirmed results of both types, success and failure, on materials of known behavior in service, it could be used in predicting the behavior of other materials. That it did confirm service results would indicate that the results are representative of the materials themselves.

SOME MECHANICAL PROPERTIES OF NICKEL, MANGANESE-NICKEL AND COPPER-NICKEL ALLOYS[1]

By William A. Mudge[2] and Leroy W. Luff[3]

Synopsis

The purpose of the investigation described in this paper was to supplement the existing mechanical data for nickel and some of its important, commercial, corrosion-resisting alloys.

New and representative average mechanical properties in tension, torsion, impact and hardness are given for pure nickel, manganese-nickel alloys containing 2.0 and 4.5 per cent of manganese, respectively, and copper-nickel alloys of the constantan type. Using these data, calculations of Poisson's ratio have been made.

Additions of manganese up to 4.63 per cent result in a definite increase in strength, toughness and hardness with practically no decrease in ductility. All materials tested show exceptionally high ratios of strength to ductility.

Engineering practice in our present industrial development demands continual improvement in materials of construction, particularly those structural materials that must be resistant to corrosion. The extensive use of nickel and its alloys for corrosion and heat-resisting purposes, such as valves for superheated steam, turbine blades, gas-engine parts, firebox and anchor bolts, is a direct result of study and application of their properties. The International Nickel Co. is particularly interested in the collection of complete, accurate information and data for nickel and its alloys in order to supplement those phases of the subject which have previously and satisfactorily been presented.[4]

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[1] Contribution from The International Nickel Co., 67 Wall Street, New York City, N. Y.

[2] Metallurgist, Huntington Works, The International Nickel Co., Huntington, W. Va.

[3] Research Associate, Huntington Works, The International Nickel Co., Huntington, W. Va.

[4] D. H. Browne and J. F. Thompson, "Physical Properties of Nickel," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 64, p. 378 (1920).

A. T. Wilgress, "Report and Appendix of the Royal Ontario Nickel Commission," Toronto (1917).

P. D. Merica, R. G. Waltenberg and A. S. McCabe, "Some Mechanical Properties of Hot-Rolled Monel Metal," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 922 (1921).

R. G. Waltenberg, "Notes on Notched Bar Impact Tests and Toughness of Monel Metal," *Chemical and Metallurgical Engineering*, Vol. 25, No. 8 (1921).

M. A. Hunter, F. M. Sebast and A. Jones, "Some Electrical Properties of Nickel and Monel Metal Wires," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 68, p. 750 (1922).

"Nickel and Its Alloys," U. S. Bureau of Standards, *Circular No. 100* (1924).

D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper," *Transactions*, Am. Soc. Steel Treating, Vol. 7, pp. 54–81, 217–249, 581–617 (1925).

C. A. Crawford, "Nickel and Monel Metal with Especial Reference to Annealing," *Transactions*, Am. Inst. Mining and Metallurgical Engrs., *Technical Publication No. 35* (1927).

The data presented in this paper include properties of commercially pure nickel, manganese-nickel alloys containing 2.00 and 4.50 per cent of manganese, and copper-nickel alloys of the constantan type.

SELECTION OF MATERIAL

The material used for this investigation was produced at the Huntington Works of The International Nickel Co., Huntington, W. Va., over a period of two and one-half years. It was selected from the regular mill production during this period and, therefore, represents typical, commercial chemical compositions and mill sizes. The selection comprises a total of forty-five different melts which were hot-rolled to fourteen different sizes of rods, varying in diameter from $\frac{11}{16}$ to 3 in., as follows:

10 melts and 9 sizes of grade "A" Nickel (pure Nickel).
16 melts and 11 sizes of grade "E" Nickel (2.00 per cent of Manganese).
9 melts and 8 sizes of grade "D" Nickel (4.50 per cent of Manganese).
9 melts and 8 sizes of Copper-Nickel (constantan) containing 45 per cent of Nickel.

TABLE I.—CHEMICAL COMPOSITION OF MATERIAL TESTED.

Material	Carbon, per cent	Manganese, per cent	Iron, per cent	Sulfur, per cent	Silicon, per cent	Copper, per cent	Nickel, per cent
Grade "A" Nickel	0.10	0.05	0.40	0.005	0.06	0.16	99.18
Grade "E" Nickel	0.11	1.89	0.52	0.005	0.05	0.18	97.22
Grade "D" Nickel	0.11	4.63	0.56	0.005	0.05	0.18	94.44
1.00 per cent Manganese Copper-Nickel	0.05	1.02	0.27	0.006	0.02	53.79	44.86
0.50 per cent Manganese Copper-Nickel	0.05	0.48	0.27	0.006	0.02	53.95	45.18
Special Reversed Percentage Copper-Nickel	0.04	0.20	0.34	0.005	0.02	44.28	55.09

To these was added one special melt of copper-nickel containing 55 per cent of nickel. This last alloy is not a regular commercial product and has been included in this paper only to show the slight variation in mechanical properties due to a difference in chemical composition of 10 per cent of both copper and nickel.

Each melt selected represents 8000 lb. of metal which was refined in a basic electric furnace, poured into 4000-lb. ingots, milled, chipped, forged and rolled to finished rods.[1]

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[1] W. L. Wotherspoon, "Refinery and Rolling Mill for Monel Metal," *Transactions*, Am. Soc. Mechanical Engrs., Vol 44, p. 975 (1922).

R. S. McBride, "Quantity Methods in the Production of Quality Metals and Alloys," *Chemical and Metallurgical Engineering*, Vol. 29, pp. 745–751 (1923).

C. A. Crawford, "Nickel and Monel Metal, with Especial Reference to Annealing," Am. Inst. Mining and Metallurgical Engrs., *Technical Publication No. 35* (1927).

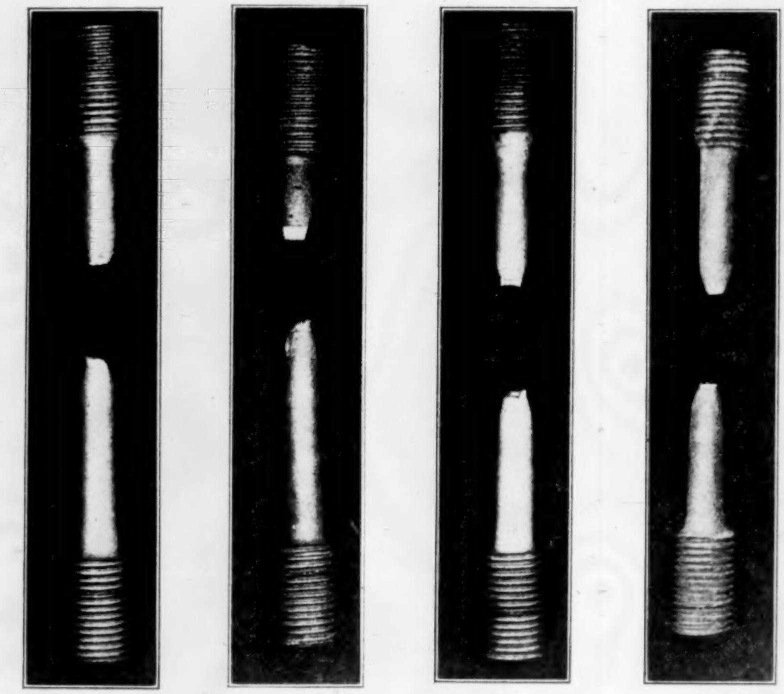
These articles give a detailed description of the different mill operations through which the materials used in this investigation were processed.

CHEMICAL COMPOSITION

Table I gives the composition of each grade of material used. Each analysis is a numerical average of all material in each of the several groups.[1]

METHODS OF TESTING

All tests were made on the different sizes of materials as hot rolled and as annealed for 4 hours at 1400° F. The tension and torsion tests were made in duplicate or triplicate and the Izod impact tests in duplicate with a few in quadruplicate.



Annealed "A" Nickel

Annealed "E" Nickel

Annealed "D" Nickel

Annealed Copper-Nickel

FIG. 1.—Typical Fractured Tension Test Specimens.

Tension and Hardness Tests.—Tension tests were made according to the procedure prescribed in the Tentative Methods of Tension

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[1] It is considered preferable, due to the small variation in chemical composition of the different melts in each grade selected for testing, and a proportionately small variation in their mechanical properties, to give a typical analysis for each grade rather than to enlarge this paper with unnecessary tabulation.

Testing of Metallic Materials (E 8 - 27 T) of the American Society for Testing Materials,[1] with a 200,000-lb. Olsen machine, using the 0.505-in. diameter, 2-in. gage length machined test specimens. Values up to the proportional limit (testing speed 0.048 in. per minute) were obtained with a Ewing extensometer, readings of stress being taken for every 0.0002-in. of elongation. Figure 1 shows typical fractures of annealed test specimens. Hardness values were determined with a 3-A Rockwell hardness tester, using the 100-kg. load ("B" Scale); a 3000-kg. Brinell machine; and a model "D" Shore scleroscope. Each hardness value is the average of at least four readings.

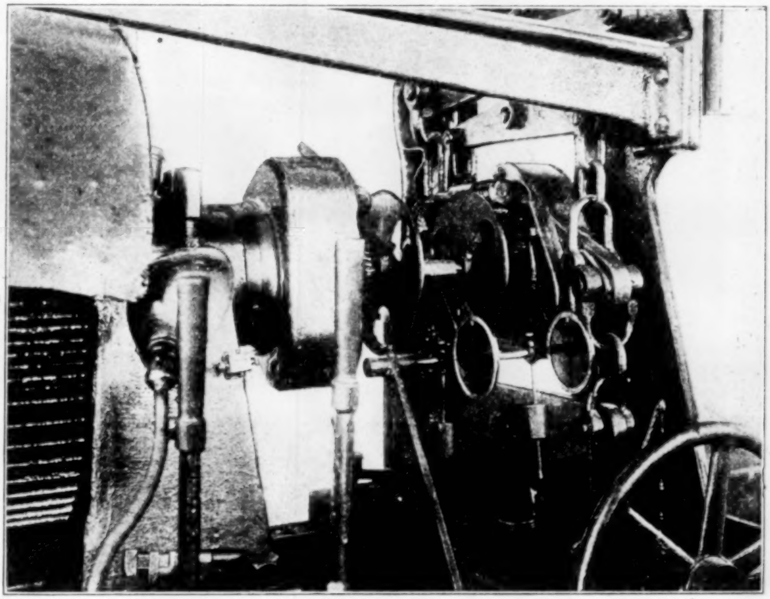

FIG. 2.—Torsion Machine with Specimen and Indicating Dials in Position.

Torsion Tests.—The torsion tests were made with a 60,000 - in-lb. Riehlé machine after the manner described in detail by Merica, Waltenberg and McCabe,[2] except for the dimensions of the test specimen. Rods up to $1\frac{1}{2}$ in. were twisted full size; the larger sizes were machined to $1\frac{3}{8}$ in. in diameter in the gage length, leaving the grip sections $1\frac{3}{4}$ in. in diameter. The machined portion was 10 in. long and was joined to the grip section by fillets of large radius; the gage

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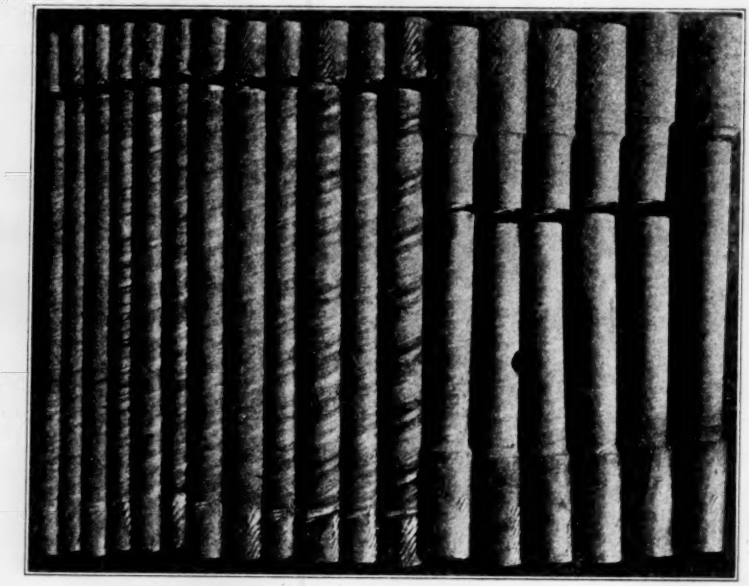
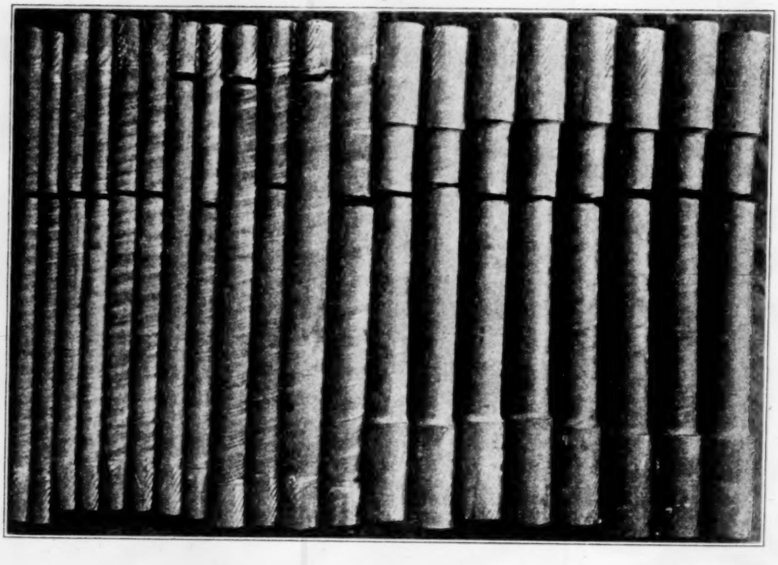
[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1067 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 725.

[2] P. D. Merica, R. G. Waltenberg and A. S. McCabe, "Some Mechanical Properties of Hot-Rolled Monel Metal," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 922 (1921).



(*a*) Annealed "E" Nickel



(*b*) Annealed Copper-Nickel

FIG. 3.—Typical Fractured Torsion Specimens.

length was 6 in. The specimen measured 18 in. overall. A twisting speed of 11 deg. per minute was used up to the yield point and of 400 deg. per minute for the breaking strength. Figure 2 shows the machine with specimen and Wissler indicating dials in position. Figure 3 shows typical annealed, fractured test specimens.

Impact Tests.—The impact data were determined with a 120-ft-lb. Olsen Izod impact machine. Test specimens were machined to the standard single-notch Izod test specimen, 1 cm. square.[1]

CALCULATION OF RESULTS

Tension Tests.—All data were calculated to correspond with, and in the manner specified by, the A.S.T.M. standards, except that an elongation of 0.01 in. in 2 in. was used to determine the yield point, since the drop of the beam is not as definite as in the case of steel.

Torsion Tests.—The following formula was used to compute the values for proportional limit and yield point:

$$S_s = \frac{M_t r}{J} \qquad \text{(1)}$$

in which M = twisting moment in inch-pounds,
r = radius of the test piece in inches, and
J = polar moment of inertia $\frac{(\pi\, r^4)}{2}$,

which reduces to the empirical formula:[2]

$$S_s = \frac{5.08\ M_t}{d^3} \qquad \text{(2)}$$

For computing the breaking strength the formula:

$$S_s = \frac{\frac{3}{4}\ M_t r}{J}$$

was used, according to Upton[3] who shows mathematically that the ultimate shearing stress in torsion is approximately three-fourths of the value as calculated from Eq. 1.

This reduces to the empirical formula:[2]

$$S_s = \frac{3.81\ M_t}{d^3} \qquad \text{(3)}$$

The calibration of the dials in radians per division, obtained by turning the head of the machine with the specimen free at the stationary jaw, through several accurately measured turns, gave a value of 0.000463 radians per division.

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[1] P. N. Menefee, Am. Soc. Steel Treating Handbook, p. V-24.

[2] International Critical Tables Vol. 2, p. 8.

[3] Upton, "Materials of Construction," p. 52.

TABLE II.—SUMMARY OF AVERAGE TENSILE AND HARDNESS PROPERTIES OF NICKEL, COPPER-NICKEL, AND MANGANESE-NICKEL ALLOYS.

Condition of Material	Material	Diameter, in.	Average Number of Tests	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Ultimate Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Rockwell Hardness, "B" Scale, 100 kg. load	Ratios of Mechanical Properties			Some Products or Quality Factors				Merit Index[a]	Modulus of Elasticity in Tension Young's Modulus lb. per sq. in.
										Ratio, Proportional Limit to Yield Point	Ratio, Yield Point to Tensile Strength	Ratio, Proportional Limit to Rockwell Hardness times 0.01	Tensile Strength times Elongation	Yield Point times Elongation	One-half sum of Yield Point and Tensile Strength times Elongation	Ratio of Yield Point to Tensile Strength times Elongation		
Hot Rolled	"A" Nickel	$\frac{3}{4}$ to 1	6	23 450	32 700	77 850	45.2	62.1	68	0.72	0.42	3.45	3 520 000	1 480 000	2 500 000	19.0	60 400	29 900 000
Annealed..		$\frac{3}{4}$ to 1	6	17 550	24 600	76 950	49.1	75.8	60	0.71	0.32	2.93	3 780 000	1 210 000	2 490 000	15.7	95 900	31 100 000
Hot Rolled		1 to 2	8	18 450	30 750	77 550	44.0	59.9	64.5	0.60	0.40	2.86	3 410 000	1 350 000	2 380 000	17.4	52 700	31 400 000
Annealed..		1 to 2	8	19 550	27 000	77 700	47.8	76.0	59.5	0.72	0.35	3.29	3 710 000	1 290 000	2 500 000	16.8	96 800	31 200 000
Hot Rolled		2 to 3	6	27 950	37 650	76 000	41.8	56.5	70	0.74	0.50	3.99	3 180 000	1 570 000	2 370 000	20.7	49 900	31 200 000
Annealed..		2 to 3	6	22 000	28 600	75 900	48.9	76.0	62	0.77	0.38	3.55	3 710 000	1 400 000	2 510 000	18.4	99 700	30 900 000
Hot Rolled		All sizes	20	22 800	33 400	77 200	43.7	59.5	67	0.68	0.43	3.40	3 370 000	1 460 000	2 420 000	18.9	53 950	30 900 000
Annealed..		All sizes	20	19 700	26 750	76 850	48.5	75.9	60.5	0.74	0.35	3.26	3 730 000	1 300 000	2 510 000	16.9	97 150	31 100 000
Hot Rolled	"E" Nickel	$\frac{11}{16}$ to 1	13	22 650	28 900	82 300	47.9	66.8	65.5	0.78	0.35	3.46	3 940 000	1 380 000	2 690 000	16.8	76 150	32 300 000
Annealed..		$\frac{11}{16}$ to 1	13	21 050	26 300	81 700	50.4	73.2	61.0	0.80	0.32	3.45	4 120 000	1 330 000	2 720 000	16.2	96 600	31 500 000
Hot Rolled		1 to 2	13	22 700	32 800	81 050	45.9	61.0	67.5	0.69	0.41	3.36	3 710 000	1 510 000	2 610 000	18.5	61 000	31 800 000
Annealed..		1 to 2	14	21 350	26 550	80 150	50.2	74.5	61.0	0.81	0.33	3.52	4 030 000	1 330 000	2 680 000	16.6	99 750	31 900 000
Hot Rolled		2 to 3	13	31 100	41 350	81 950	43.2	62.5	74.0	0.76	0.51	4.20	3 540 000	1 780 000	2 670 000	21.8	66 100	31 000 000
Annealed..		2 to 3	12	23 750	30 800	80 400	48.2	75.2	64.0	0.77	0.38	3.73	3 860 000	1 490 000	2 680 000	18.5	101 500	31 900 000
Hot Rolled		All sizes	39	25 500	34 350	81 800	45.6	63.5	69.0	0.74	0.42	3.70	3 740 000	1 570 000	2 650 000	18.6	67 400	31 800 000
Annealed..		All sizes	39	22 000	27 500	80 700	49.7	74.2	63.0	0.79	0.34	3.48	4 010 000	1 380 000	2 690 000	17.1	99 450	31 600 000

[a] Merit Index = $\dfrac{\dfrac{T. + P.L.}{2} \times e}{100 - R.A.}$; where:

$T.$ = tensile strength in pounds per square inch;
$P.L.$ = proportional limit in pounds per square inch;
e = elongation in 2 in., per cent; and
$R.A.$ = reduction of area.

TABLE II.—SUMMARY OF AVERAGE TENSILE AND HARDNESS PROPERTIES OF NICKEL, COPPER-NICKEL, AND MANGANESE-NICKEL ALLOYS—(*Continued*).

Condition of Material	Material	Diameter in.	Average Number of Tests	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Ultimate Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Rockwell Hardness,"B" Scale, 100 kg load	Ratios of Mechanical Properties			Some Products or Quality Factors				Merit Index[a]	Modulus of Elasticity in Tension Young's Modulus lb. per sq. in.
										Ratio, Proportional Limit to Yield Point	Ratio, Yield Point to Tensile Strength	Ratio, Proportional Limit to Rockwell Hardness times 0.01	Tensile Strength Times Elongation	Yield Point Times Elongation	One-half sum of Yield Point and Tensile Strength times Elongation	Ratio of Yield Point to Tensile Strength times Elongation		
Hot Rolled	"D" Nickel	$\frac{11}{16}$ to 1	6	32 300	37 000	89 650	39.5	57.8	73.5	0.87	0.41	4.40	3 540 000	1 460 000	2 500 000	16.3	57 100	31 900 000
Annealed..		$\frac{11}{16}$ to 1	6	25 000	33 200	87 300	48.1	70.6	69	0.75	0.38	3.62	4 200 000	1 600 000	2 900 000	18.2	91 850	30 800 000
Hot Rolled		1 to 2	7	24 150	34 050	86 600	42.8	52.9	71	0.71	0.39	3.40	3 710 000	1 460 000	2 580 000	16.8	50 300	32 800 000
Annealed..		1 to 2	6	27 450	33 100	88 800	49.5	72.0	69.5	0.83	0.37	3.95	4 400 000	1 640 000	3 020 000	18.4	102 700	31 200 000
Hot Rolled		2 to 3	6	33 450	45 350	83 200	36.2	46.9	78	0.74	0.54	4.29	3 010 000	1 640 000	2 330 000	19.6	39 750	32 100 000
Annealed..		2 to 3	7	25 300	31 800	82 150	49.1	74.2	66	0.80	0.39	3.83	4 030 000	1 560 000	2 800 000	18.0	102 200	32 200 000
Hot Rolled		All sizes	19	31 350	38 550	86 500	39.7	52.6	77	0.81	0.45	4.07	3 430 000	1 530 000	2 480 000	17.7	49 350	32 300 000
Annealed..		All sizes	19	25 850	32 650	85 800	48.9	72.4	68	0.79	0.38	3.80	4 200 000	1 600 000	2 900 000	19.0	98 900	31 500 000
Hot Rolled	Copper Nickel	$\frac{3}{4}$ to 1	6	21 950	31 500	68 050	44.5	77.8	61.5	0.70	0.46	3.57	3 030 000	1 400 000	2 220 000	20.6	90 200	25 600 000
Annealed..		$\frac{3}{4}$ to 1	7	21 700	29 200	68 250	46.4	78.8	58	0.74	0.43	3.74	3 170 000	1 360 000	2 260 000	19.9	98 500	24 600 000
Hot Rolled		1 to 2	7	18 700	33 500	67 000	42.1	75.6	59.5	0.56	0.50	3.14	2 820 000	1 410 000	2 120 000	21.0	73 900	24 600 000
Annealed..		1 to 2	6	17 800	25 400	64 900	46.1	76.5	51.5	0.70	0.39	3.46	2 990 000	1 170 000	2 080 000	18.0	81 100	24 900 000
Hot Rolled		2 to 3	7	22 950	39 550	68 300	41.1	76.2	66	0.58	0.58	3.48	2 810 000	1 680 000	2 220 000	23.8	81 100	24 500 000
Annealed..		2 to 3	7	20 850	26 400	66 400	45.6	78.3	54	0.79	0.40	3.86	3 030 000	1 200 000	2 120 000	18.1	91 650	24 900 000
Hot Rolled		All sizes	20	21 150	35 000	67 750	42.5	76.5	62.5	0.60	0.52	3.38	2 880 000	1 490 000	2 180 000	22.0	80 400	24 900 000
Annealed..		All sizes	20	20 250	27 100	66 600	46.0	77.9	54.5	0.75	0.41	3.72	3 060 000	1 250 000	2 160 000	18.8	90 350	24 800 000
Hot Rolled	Special Reversed Percentage of Copper-Nickel	1	2	13 500	29 850	65 850	46.3	79.3	58	0.45	0.45	2.33	3 050 000	1 380 000	2 220 000	20.8	88 750	26 900 000
Annealed..		1	2	19 250	25 650	64 250	46.0	78.9	51.5	0.75	0.40	3.74	2 960 000	1 180 000	2 070 000	18.4	91 000	26 600 000

[a] Merit Index = $\dfrac{\dfrac{T.+P.L.}{2}\times e}{100-R.A.}$; where: $T.$ = tensile strength in pounds per square inch;
$P.L.$ = proportional limit in pounds per square inch;
e = elongation in 2 in., per cent; and
$R.A.$ = reduction of area.

The yield point in torsion has been defined as the stress at which the linear shear of the outermost fiber was 0.01 in. per inch of gage length. Therefore, in order to give values comparable with those of the tension tests[1] and to read the values of the yield points in inch-pounds directly on the beam of the machine, the difference in dial readings corresponding to the 0.06-in. linear shear was computed from the formula:

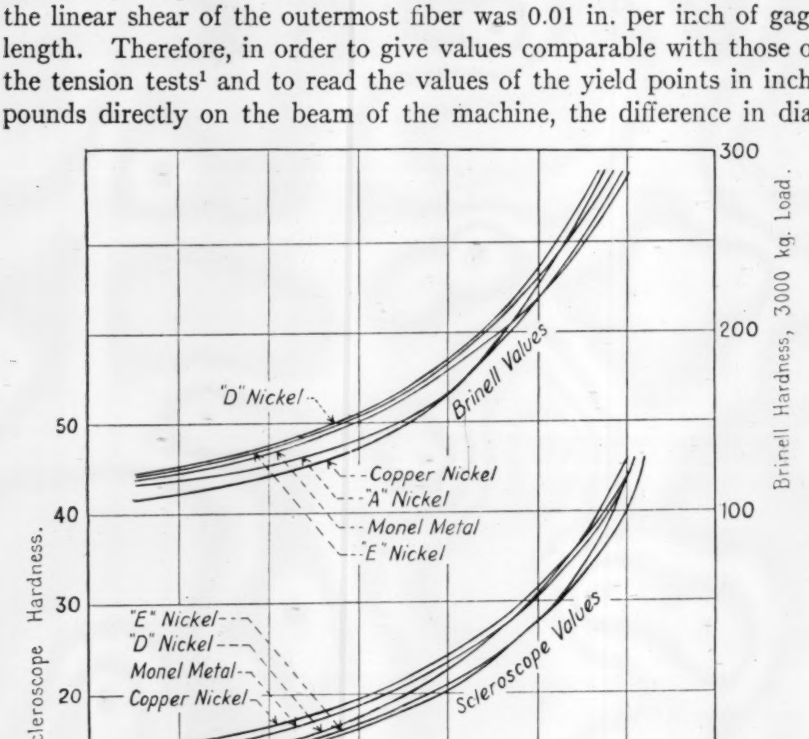

FIG. 4.—Curves for Conversion of Rockwell Hardness Values to Shore and Brinell Values.

$$d = \frac{0.01 \times 6}{\text{diameter of rod in inches} \times 0.000463} \ldots\ldots\ldots\ldots (4)$$

in which d = difference in readings of the dials and
0.000463 = radians per division of the dials.

The modulus of elasticity in shear was calculated from the formula:[2]

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[1] P. D. Merica, R. G. Waltenberg and A. S. McCabe, "Some Mechanical Properties of Hot-Rolled Monel Metal," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 922 (1921).

[2] Kent, "*Mechanical Engineers Pocket Book*," 9th Edition, p. 334.

TABLE III.—SUMMARY OF AVERAGE TORSIONAL PROPERTIES OF NICKEL, COPPER-NICKEL AND MANGANESE-NICKEL ALLOYS.

Condition of Material	Material	Diameter, in.	Average Number of Tests	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Breaking Strength, lb. per sq. in.	Total Twist, deg.	Angle of Twist, deg. per in.	Shear (twists per inch circumference), in per inch	Ratios of Mechanical Properties			Merit Index[a]	Modulus of Elasticity in Shear, lb. per sq. in.
										Ratio, Proportional Limit in Shear to Proportional Limit in Tension	Ratio, Yield Point in Shear to Yield Point in Tension	Ratio, Breaking Strength in Shear to Breaking Strength in Tension		
Hot Rolled	"A" Nickel	$\frac{3}{4}$ to 1	6	27 300	32 800	66 250	6 042	406	3.07	1.16	0.98	0.85	72 200	11 580 000
Annealed		$\frac{3}{4}$ to 1	6	9 850	14 450	61 650	5 835	396	3.00	0.55	0.59	0.80	54 000	11 020 000
Hot Rolled		1 to 2	8	23 350	28 400	65 200	3 702	257	2.81	1.31	0.92	0.84	63 400	11 270 000
Annealed		1 to 2	8	12 850	16 300	64 550	3 740	254	2.81	0.70	0.66	0.83	54 400	10 240 000
Hot Rolled		2 to 3	6	17 100	22 100	64 800	2 872	201	2.39	0.63	0.59	0.85	49 000	11 130 000
Annealed		2 to 3	6	11 750	15 450	67 600	3 508	238	2.82	0.54	0.54	0.89	56 200	11 920 000
Hot Rolled		All sizes	20	22 650	27 850	65 400	4 155	285	2.76	1.06	0.84	0.85	61 700	11 320 000
Annealed		All sizes	20	11 600	15 500	64 600	4 299	291	2.87	0.61	0.60	0.84	54 800	10 980 000
Hot Rolled	"E" Nickel	$\frac{11}{16}$ to 1	12	15 550	19 900	66 850	5 728	394	2.88	0.69	0.69	0.81	59 350	9 850 000
Annealed		$\frac{11}{16}$ to 1	12	13 650	18 100	65 250	5 840	395	2.89	0.68	0.70	0.80	57 000	10 420 000
Hot Rolled		1 to 2	12	27 050	33 300	64 100	3 230	222	2.62	1.21	1.00	0.79	59 800	11 990 000
Annealed		1 to 2	12	15 900	18 450	63 400	3 525	237	2.80	0.75	0.70	0.79	55 550	12 090 000
Hot Rolled		2 to 3	12	17 900	23 750	65 850	2 540	196	2.35	0.60	0.60	0.81	49 300	11 540 000
Annealed		2 to 3	12	12 850	16 400	70 900	3 710	259	3.11	0.58	0.56	0.88	64 950	11 810 000
Hot Rolled		All sizes	36	20 150	25 650	65 600	3 832	271	2.62	0.83	0.76	0.80	56 150	11 130 000
Annealed		All sizes	36	14 150	17 650	65 650	4 360	297	2.93	0.67	0.65	0.82	59 150	11 440 000
Hot Rolled	"D" Nickel	$\frac{11}{16}$ to 1	6	21 000	25 250	72 700	5 520	380	2.68	0.65	0.68	0.81	62 600	9 650 000
Annealed		$\frac{11}{16}$ to 1	5	18 800	24 300	68 100	4 913	345	2.51	0.78	0.74	0.78	55 300	10 640 000
Hot Rolled		1 to 2	6	23 300	28 200	68 350	2 952	203	2.21	0.98	0.83	0.80	50 600	11 340 000
Annealed		1 to 2	6	18 750	22 800	66 650	2 900	200	2.18	0.69	0.69	0.75	46 600	11 080 000
Hot Rolled		2 to 3	6	18 700	27 800	62 800	1 947	163	1.95	0.56	0.62	0.75	40 050	10 710 000
Annealed		2 to 3	6	13 950	19 350	71 050	3 762	260	3.12	0.55	0.60	0.86	66 650	11 100 000
Hot Rolled		All sizes	18	21 000	27 100	67 900	3 472	249	2.28	0.73	0.71	0.79	51 100	10 570 000
Annealed		All sizes	17	17 050	22 000	68 650	3 796	264	2.61	0.67	0.67	0.80	56 250	10 940 000
Hot Rolled	Copper-Nickel	$\frac{3}{4}$ to 1	6	16 750	21 800	50 500	8 755	593	4.46	0.76	0.69	0.74	75 200	8 650 000
Annealed		$\frac{3}{4}$ to 1	6	17 350	21 400	49 600	7 695	520	3.91	0.81	0.74	0.73	65 400	9 150 000
Hot Rolled		1 to 2	6	20 400	23 400	47 400	4 527	305	3.62	1.05	0.68	0.70	62 100	10 660 000
Annealed		1 to 2	6	12 100	15 850	47 000	4 638	314	3.75	0.68	0.63	0.72	55 400	9 400 000
Hot Rolled		2 to 3	6	15 800	22 500	48 450	3 950	271	3.24	0.69	0.60	0.71	51 800	9 070 000
Annealed		2 to 3	6	12 900	17 200	47 150	3 603	251	3.01	0.61	0.65	0.71	45 200	9 080 000
Hot Rolled		All sizes	18	17 650	22 600	48 800	5 743	390	3.77	0.83	0.66	0.72	63 050	9 460 000
Annealed		All sizes	18	14 100	18 150	47 900	5 312	362	3.55	0.70	0.67	0.72	55 300	9 210 000
Hot Rolled	Special Reversed Percentage of Copper-Nickel	1	2	16 850	21 750	46 150	7 300	495	4.37	1.25	0.73	0.70	68 800	9 700 000
Annealed		1	2	14 750	19 100	48 300	6 330	431	3.79	0.77	0.75	0.75	58 800	9 420 000

[a] Merit Index = $\dfrac{\dfrac{B.S. + P.L.}{2} \times \text{Shear}}{2}$; where: $B.S.$ = breaking strength in pounds per square inch;
$P.L.$ = proportional limit in pounds per square inch.

TABLE IV.—AVERAGE IZOD IMPACT DATA.

Results are averages of two tests, except as noted.

Diameter, of Rod Tested, in.	"A" Nickel				"E" Nickel				"D" Nickel			
	Hot Rolled		Annealed		Hot Rolled		Annealed		Hot Rolled		Annealed	
	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]
$\frac{11}{16}$	..	..	..	..	108	*C*	87.5	*A*	..	..	..	..
$\frac{3}{4}$	102	*C*	72	*A* (coarse)	109[c]	*C*	89.5	*A*	..	..	..	..
$\frac{7}{8}$	103	*C*	81	*A*	103	*C*	92	*A*	110[c]	*C*	69[c]	Between *A* and *B*
1	92.5	Between *C* and *D*	84[b]	*A*	94[c]	Between *C* and *D*	89	*A*	..	..	..	..
$1\frac{1}{8}$	..	..	..	..	..	..	..	..	113	Between *B* and *C*	100.5	*A*
$1\frac{1}{4}$	94	*C*	81	*A* (coarse)	93[c]	*C*	91	*A*	97	Between *C* and *D*	106	*A*
$1\frac{5}{16}$	..	..	..	..	95	*C*	86	*A*	..	..	..	..
$1\frac{3}{8}$	100	*C*	85	*A*	100.5	Between *B* and *C*	90.5	*A*	..	..	..	..
$1\frac{1}{2}$	..	..	..	..	105[c]	Between *B* and *C*	95	*A*	106	*C*	103	*A*
$1\frac{3}{4}$	..	..	..	..	..	..	..	..	..	..	..	..
$1\frac{7}{8}$	98	*C*	82	*A*	..	..	..	..	..	..	..	..
2	..	..	..	..	106[c]	*C*	87[b]	*A*	105	*C*	88	*A*
$2\frac{1}{2}$	..	..	..	..	> 120[c]	*C*	89.5	*A*	> 120	Between *C* and *D*	96	*A*
3	90.5	*C*	72.5	*A* (coarse)	> 120[c]	*C*	79	*A*	> 120	*C*	99	*A*

Diameter of Rod Tested, in.	1 Per Cent Manganese Copper-Nickel				0.5 Per Cent Manganese Copper-Nickel				Special Reversed Percentage Copper-Nickel			
	Hot Rolled		Annealed		Hot Rolled		Annealed		Hot Rolled		Annealed	
	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]	Foot Pounds	Fracture[a]
$\frac{11}{16}$	..	..	..	..	..	..	..	..	..	..	..	..
$\frac{3}{4}$	87	Between *B* and *C*	80.5	*A*	..	..	..	..	..	..	..	..
$\frac{7}{8}$	104	*B*	88	*A*	..	..	..	..	..	..	..	..
1	96	*B*	85	*A*	..	..	..	..	95	*B*	80.5	*A*
$1\frac{1}{8}$	..	..	..	..	..	..	..	..	..	..	..	..
$1\frac{1}{4}$	87	*C*	83.5	*A*	..	..	..	..	..	..	..	..
$1\frac{5}{16}$	..	..	..	..	..	..	..	..	..	..	..	..
$1\frac{3}{8}$	..	..	..	..	..	..	..	..	..	..	..	..
$1\frac{1}{2}$	96	*B*	85.5	*A*	..	..	..	..	..	..	..	..
$1\frac{3}{4}$	90	*B*	62	*A*	..	..	..	..	..	..	..	..
$1\frac{7}{8}$	..	..	..	..	..	..	..	..	..	..	..	..
2	..	..	..	..	96[b]	*B*	105.5[b]	*A*	..	..	..	..
$2\frac{1}{2}$	106	*C*	86	*A*	..	..	..	..	..	..	..	..
3	103	*C*	79.5	*A*	..	..	..	..	..	..	..	..

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[a] Under fracture, the following letters designate appearance of sample after test:
A, Failed to fracture.
B, Incipient fracture.
C, Partial fracture — slight tearing.
D, Partial fracture — tearing about one-third through the piece.
Where a value greater than 120 ft-lb. was obtained, the pendulum did not move after striking the specimen.

[b] Single test.

[c] Four tests.

$$E_s = \frac{Tl}{\theta\, d^4}\left(\frac{32}{\pi}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which T = beam load in inch-pounds
l = length between jaws in inches
d = diameter of rod in inches
θ = angle of twist in radians measured on arc of radius = 1 (or $0.000463 \times$ number of divisions $\times \frac{1}{\text{radius}}$).

Impact and Hardness Tests.—The impact value is a direct scale reading. Although Rockwell "B" scale, Shore scleroscope and Brinell (3000-kg.) hardness values were determined for all materials only the Rockwell values have, for simplicity, been tabulated. These are with the tension test data in Table II. Conversions to scleroscope and Brinell values may readily be made through the use of the curves given in Fig. 4.

PRESENTATION OF DATA

Tables II, III and IV give complete, average and summary values in tension and hardness, in torsion and in impact, respectively, for the different grades of materials and sizes of rods.

GENERAL DISCUSSION

Without exception, the break in the stress-strain curve of the tension tests was sharp and distinct. This was not altogether true in torsion, where, as noted by McAdam[1] for copper-nickel and cold-rolled Monel metal, a straight-line graph was not obtained. As the hardness of the materials increase, a more nearly straight-line graph was obtained. Numerical values are given, however, since the variation from the linear was slight. Our values and those obtained by McAdam for the nickel-bearing materials with which he worked are substantially in accord.

Invariably, in the impact tests, the striker edge of the pendulum in bending over the specimen gouged a chip along the metal surface opposed to it, thus dissipating a portion of the energy in that work and accounting, in part, for variations noted in some of the tests. Since the Izod impact test is primarily intended to measure the energy necessary to fracture a sample, these values cannot be considered empirical and must be taken as an indication of toughness. Not one of the specimens broke under the blow of the pendulum so that most of the energy absorbed was used in bending over the specimen.

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[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper," *Transactions*, Am. Soc. Steel Treating, Vol. 7, p. 56 (1928).

The effect of manganese, up to 4.63 per cent, upon the mechanical properties of nickel is shown in Fig. 5. The increase in strength with practically no decrease in ductility is definite.

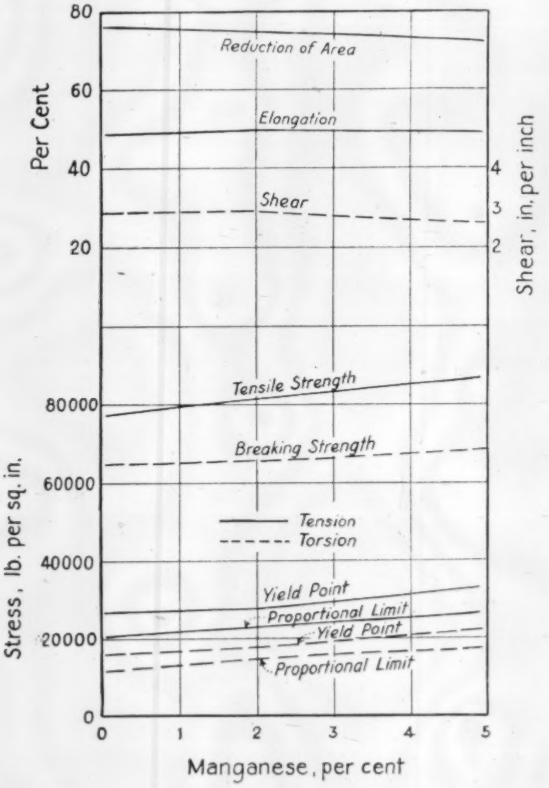

FIG. 5.—Effect of Manganese Upon the Tensile and Torsional Properties of Nickel.

The normal variations in mill practice are more clearly indicated by the torsional properties of the full-sized hot-rolled rods than by the tensile properties. The five sizes between $1\frac{1}{8}$ and $1\frac{1}{2}$ in. in diameter of all grades of material have torsional proportional limits and yield points equal to, or greater than, the corresponding values in tension. This is natural since the torsion test measures surface deformation under load more accurately than the tension test and therefore accentuates the small amount of cold work due to a lower finishing temperature on the mill. This condition disappears completely in the tests on the annealed rods.

Four quality factors and the Chandler merit index have been calculated from the tension-test data. A similar merit index formula was devised for the torsional data (see Table III). This formula arbitrarily uses a denominator 2 in order to give a figure proportionately lower than the corresponding tensile value. These quality factors indicate energy required to rupture and, when used as a supplement to the impact values, determine whether or not the metal is of standard quality.

Poisson's ratio, λ, has been calculated from the tensile and torsional moduli of Tables II and III, and has the following values for the different grades of material:

Grade "A" Nickel (pure nickel)	0.359
Grade "E" Nickel (2.0 per cent of manganese)	0.356
Grade "D" Nickel (4.5 per cent of manganese)	0.339
Copper-Nickel (constantan type)	0.376
Special Copper-Nickel (reversed percentages)	0.357

These values agree rather closely with the value of 0.33 reported by Benton[1] for monel metal and nickel.

CONCLUSIONS

1. New and representative average mechanical properties in tension, torsion, impact and hardness are given for typical mill sizes of five commercial products of nickel and nickel alloys with manganese and with copper. These materials have an exceptionally high ratio of strength to ductility and in this respect surpass most steels and other non-ferrous products.

2. Additions of manganese up to 4.63 per cent result in a definite increase in strength, toughness and hardness with practically no decrease in ductility.

3. In order of decreasing strength and increasing ductility the products may be classified as follows:

Grade "D" Nickel (4.5 per cent of manganese).
Grade "E" Nickel (2.0 per cent of manganese).
Grade "A" Nickel (pure nickel).
Copper-Nickel (constantan type).

Acknowledgment.—The authors gratefully acknowledge the sympathetic and personal interest of Mr. Arthur S. Shoffstall, General Manager of the Huntington Works of the International Nickel Co., in this work.

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[1] J. R. Benton, *Physical Review*, Vol. 12, p. 36.

DISCUSSION

Mr. Mochel.

MR. N. L. MOCHEL[1] (*presented in written form*).—It is observed in referring to Table I of the paper that all six materials were of low carbon content, suitable for cold drawing purposes. The three nickels are approximately uniform in composition except in manganese content, and it is shown in Table II that increased tensile properties are secured from increased manganese content.

Several years ago the writer made some tests on four lots of nickel of varying carbon and silicon contents, furnished by Mr. Suhl of the International Nickel Co.:

Material	Chemical Composition, per cent							
	Carbon	Manganese	Sulfur	Silicon	Iron	Copper	Cobalt	Nickel
M-24	0.09	0.09	0.007	0.02	0.54	0.22	0.38	98.70
M-25	0.16	0.29	0.007	0.45	0.70	0.19	0.32	97.86
M-29	0.28	0.29	0.01	nil	0.18	0.22	0.29	98.70
M-30	0.29	0.30	0.01	0.53	0.46	0.23	0.34	97.80

The material was furnished in $1\frac{3}{8}$-in. round hot-rolled bars, and tests were made in the condition as received. Drop forgings were particularly in mind at the time, and $\frac{3}{4}$-in. round pieces were forged from the bars in one heat. Tests were also made in the forged state. Other forged pieces were annealed at 750, 1400 and 1900° F. and tested. The average results are given below:

Material	Condition	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent	Izod Impact Value, ft-lb.	Brinell Hardness, 3000-kg. load, 10-mm. ball	Rockwell Hardness, 100-kg. load, $\frac{1}{16}$-in. ball
M-24	$1\frac{3}{8}$-in. hot-rolled	21 500	31 000	74 500	45.0	61.1	84	109	74
	As forged	32 000	41 000	78 000	31.0	58.0	95	125	74
	Annealed at 750° F.	37 000	40 000	78 000	35.0	66.1	..	..	..
	Annealed at 1400° F	22 000	27 750	73 200	44.0	74.9	..	..	..
	Annealed at 1900° F	14 000	17 500	71 000	49.0	72.5	..	..	..
M-25	$1\frac{3}{8}$-in hot-rolled	34 500	45 700	88 000	41.8	61.2	105	150	80
	As forged	44 000	47 000	86 500	37.0	63.5	120	158	81
	Annealed at 750° F.	37 500	44 000	88 000	43.0	67.0	..	..	..
	Annealed at 1400° F.	25 000	34 500	80 000	42.0	68.0	..	..	..
	Annealed at 1900° F.	11 500	19 500	77 700	54.0	73.3	..	..	..
M-29	$1\frac{3}{8}$-in. hot-rolled	33 000	50 000	97 000	48.0	55.2	114	163	89
	As forged	46 500	55 000	105 000	38.5	61.6	..	170	96
	Annealed at 750° F.	35 000	52 500	101 500	44.0	67.5	..	..	..
	Annealed at 1400° F.	29 250	42 000	83 000	43.0	60.1	..	..	..
	Annealed at 1900° F	20 500	47 000	95 800	49.5	63.0	..	..	..
M-30	$1\frac{3}{8}$-in. hot-rolled	40 000	47 500	96 000	47.0	52.5	116	174	89
	As forged	47 500	61 500	104 000	42.0	56.5	..	..	..
	Annealed at 750° F.	48 000	57 500	100 500	42.0	55.7	..	..	..
	Annealed at 1400° F.	20 000	41 000	80 000	47.0	62.6	..	..	..
	Annealed at 1900° F.	16 500	46 000	92 000	48.0	60.5	..	..	..

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[1] Metallurgical Engineer, Westinghouse Electric and Manufacturing Co., Philadelphia, Pa.

Mr. Mochel.

The results of some tests made on a lot of $\frac{7}{8}$-in. round cold-drawn nickel bars may also be of interest. The composition was as follows:

Carbon, per cent	Manganese, per cent	Phosphorus, per cent	Sulfur, per cent	Silicon, per cent	Copper, per cent	Iron, per cent	Aluminum, per cent	Cobalt, per cent	Nickel, per cent
0.10	0.16	0.011	nil	0.026	0.17	0.128	nil	0.39	99.01

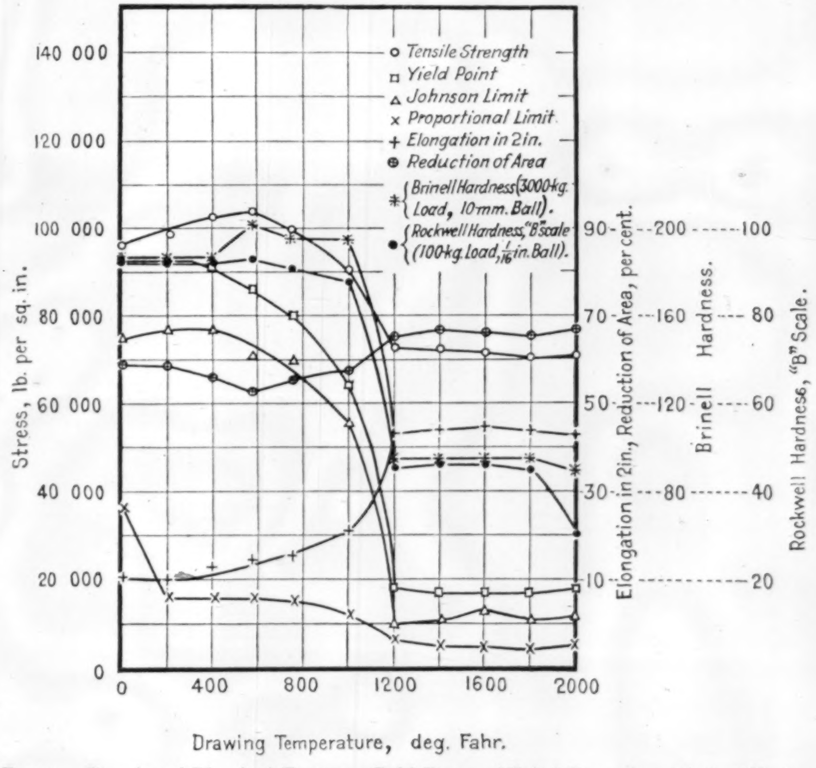
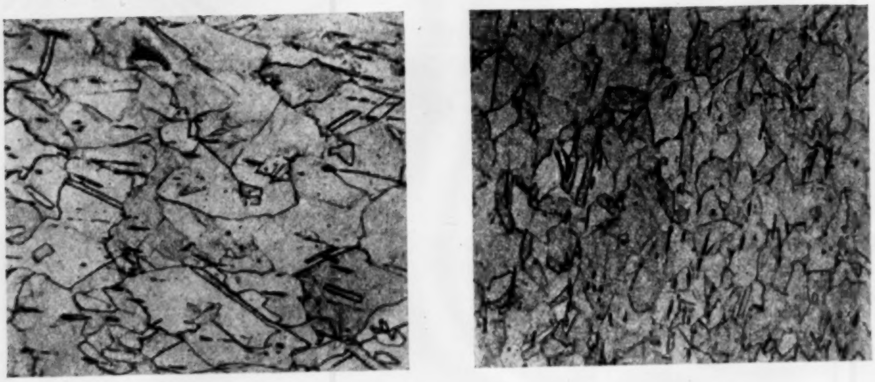


FIG. 1.—Results of Physical Tests on Cold-Drawn Nickel Bars, Annealed at Various Temperatures.

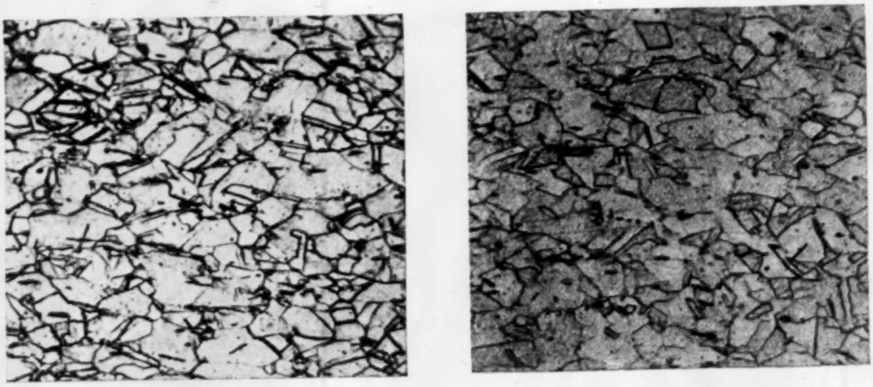
The results of eight tension tests on the material in the cold-drawn condition are given below:

	MINIMUM	MAXIMUM	AVERAGE
Proportional Limit, lb. per sq. in.	32 000	50 000	41 400
Johnson Limit, lb. per sq. in.	65 000	75 000	73 200
Yield Point, lb. per sq. in.	91 000	97 000	94 000
Tensile Strength, lb. per sq. in.	96 000	103 000	99 000
Elongation in 2 in., per cent	16.0	21.0	19.0
Reduction of Area, per cent	67.0	70.0	68.0
Brinell Hardness	179	196	190



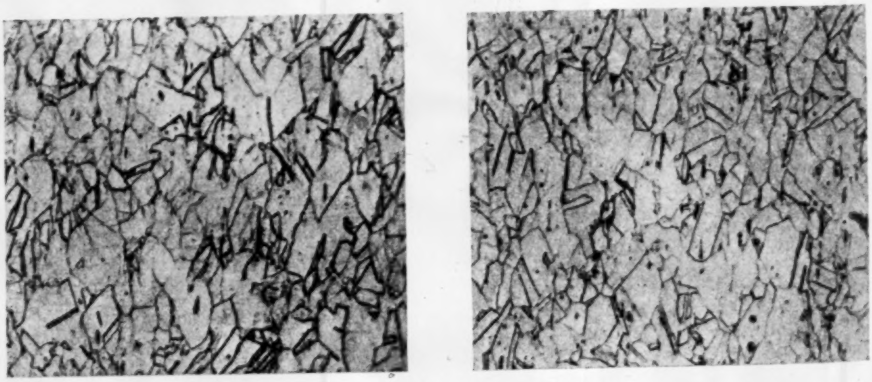
(a) Cold Drawn.

(b) Annealed at 212° F.



(c) Annealed at 400° F.

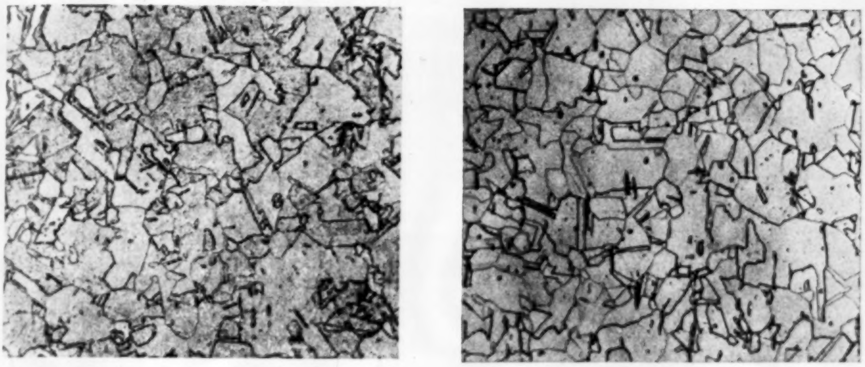
(d) Annealed at 575° F.



(e) Annealed at 750° F.

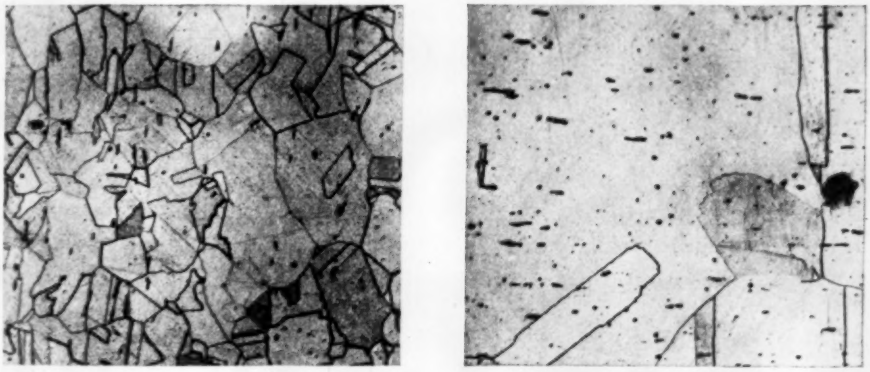
(f) Annealed at 1000° F.

FIG. 2.—Photomicrographs of Cold-Drawn Nickel, Annealed at Various Temperatures (× 100).



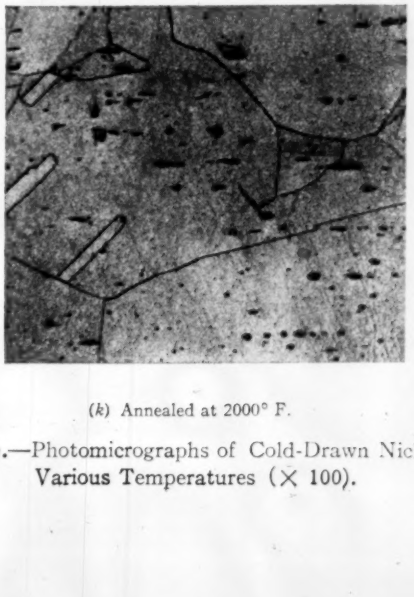
(*g*) Annealed at 1200° F.

(*h*) Annealed at 1400° F.



(*i*) Annealed at 1600° F.

(*j*) Annealed at 1800° F.



(*k*) Annealed at 2000° F.

FIG. 2. (*Continued*).—Photomicrographs of Cold-Drawn Nickel, Annealed at Various Temperatures (× 100).

Mr. Mochel. Pieces from one of the bars were annealed at temperatures from 200 to 2000° F., and the pieces used for tension, Brinell and Rockwell tests. Results are shown in the accompanying Fig. 1. Photomicrographs of the material at a magnification of 100 are also reproduced in the accompanying Fig. 2.

Grade "A" Nickel is one of the five materials being investigated by the Joint Research Committee on the Effect of Temperature on the Properties of Metals. In the report of this committee last year[1] the coefficient of expansion for this material was given. Mr. McVetty of our company has recently completed short-time tests at high temperatures on the material, and the results have been filed with the committee. Creep tests have been in progress for some time.

Mr. McAdam. MR. D. J. MCADAM, JR.[2]—In the table of compositions I find a constantan with about 1 per cent of manganese and another with about 0.5 per cent of manganese. In the table of physical properties, however, I find physical properties for but one kind of constantan. I should like to ask the authors which of the two alloys has these physical properties.

I note, also, that in what the authors call the "reversed" alloy the percentage of manganese is rather low. I had the opportunity, a few years ago, to test an alloy of this composition, kindly supplied by the International Nickel Co. for some fatigue tests. The physical properties of that alloy agreed very well with those reported in the paper. I should like to ask what would be the effect of using a larger percentage of manganese in this alloy.

In tests at the Naval Experiment Station on nickel-copper alloys, ranging in composition between constantan and commercial nickel, we found that there is some variation in physical properties due to variation in carbon content and possibly to variation in the manganese and nickel, but the general impression obtained is that for alloys within this range of composition the variations in strength and endurance limit are comparatively little.

Mr. Mudge. MR. WILLIAM A. MUDGE.[3]—The alloys with high and low carbon and silicon, respectively, the properties of which Mr. Mochel has been kind enough to give, I consider a very welcome addition to the paper. The alloys were made at the Huntington Works of the International Nickel Co., and this, I am sure, is the first time that the results of the tests which the Westinghouse Electric and Manufacturing Co. made have been published.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 153 (1927).

[2] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

[3] Metallurgist, Huntington Works, The International Nickel Co., Huntington, W. Va.

Mr. Mudge.

The question raised by Mr. McAdam regarding the difference in properties between the two copper-nickel alloys, one having 0.5 per cent of manganese and the second having 1 per cent of manganese, may be answered as follows: We found such a slight difference in the mechanical properties of these two alloys that we decided to group them, in the average, under one heading, exactly the same as was done in the case of Table I for chemical analyses, where, in order to avoid unnecessary tabulation of 45 different melts, the values for 6 averages only were reported.

Mr. McAdam's second question, regarding the possibility of increasing the strength factors by an addition of manganese, as high perhaps as 4 or 5 per cent, to the constantan alloys, is something which I cannot answer because they are alloys which we do not produce and I have not had a chance to test them.

WEAR AND MECHANICAL TESTS OF SOME RAILROAD BEARING BRONZES[1]

BY H. J. FRENCH[2]

SYNOPSIS

Copper-tin-lead alloys, both chill-cast and sand-cast, were tested under friction, without lubrication, at temperatures from 70 to 350° F. (20 to 175° C.); under friction in the presence of a lubricant at 70° F. (20° C.) and under tension, single-blow impact, and repeated pounding at temperatures from 70 to 600° F. (20 to 315° C.).

Generally, chill-cast bronzes were less resistant to wear and notched-bar impact than sand-cast bronzes, but more resistant to pounding and tension.

Increase in lead, particularly in the range 0.25 to 10 per cent, improved wearing properties but lowered the resistance to pounding, impact, and tension. Increase in tin improved all properties. Increase in temperature of test generally decreased resistance to tension, impact, pounding and wear, the changes being most marked from 350 to 600° F. (175 to 315° C.).

INTRODUCTION

This paper is an abstract of a more complete report[3] relating to the wearing properties and the mechanical properties of railroad bearing bronzes. Due to space limitations it is necessary to restrict the following discussion to important observations and the general conclusions drawn from the results of the experiments.

The need for a study of bearing bronzes was recognized about 1920 by certain of the executives of the Bostwick-Lyon Bronze Co., Waynesboro, Pa., who felt that such wide variations in the specifications of different carriers for bearing metals, as are illustrated in Fig. 1, were against the interests of the carriers as well as those of the metal manufacturers. During the latter part of 1925, subsequent to the merger of the interests of the Bostwick-Lyon Bronze Co. with those of the Chicago Bearing Metal Co., Chicago, Ill., work was begun under the research associate plan at the U. S. Bureau of Standards. During the summer of 1927, while the tests were still in progress, the interests of the Chicago Bearing Metal Co. were merged with those of the Magnus Co., Chicago, Ill., which continued to support the work until September, 1927. The final phases of the program, including most of the mechanical tests, were completed subsequent to this time by the staff of the Bureau of Standards.

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[1] Published with the approval of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] Forthcoming Bureau of Standards *Technologic Paper*, entitled "Wear and Mechanical Properties of Railroad Bearing Bronzes at Different Temperatures," by H. J. French, S. J. Rosenberg, W. LeC. Harbaugh and H. C. Cross.

The tests were started largely through the active efforts of Messrs. E. R. Darby, formerly chief metallurgist, and W. M. Corse, formerly consulting metallurgist, Chicago Bearing Metal Co., who acted in an advisory capacity throughout the investigation. Cooperation with the Magnus Co. was largely through R. J. Shoemaker, engineer of tests.

MATERIALS TESTED

Two groups of copper-tin-lead alloys were studied. In the first the copper-tin ratio was approximately 92.5 to 7.5 and the proportion of lead was varied from 0.25 to about 25 per cent. In the second group the copper-lead ratio was approximately 84 to 16 and the proportion of tin was varied from a trace to about 10 per cent.

The tests were made on specimens machined from "sticks" about 12 in. long and $2\frac{1}{2}$ in. in diameter. Sticks of each composition were cast in sand molds and a second set in metal molds. The structures of representative specimens of some of the alloys are shown in Figs. 2 and 3. Variations in structure and properties are to be expected from the outside to the center of the castings so that the specimens for metallographic examination and the mechanical tests were all taken from areas representative of the metal near the surfaces of the wear test specimens. The latter were all cut from similar positions in the various test castings.

The steel specimens with which the bronzes were in contact in the wear tests were cut from two used railroad car axles. The first of these (steel *A*) contained 0.37 per cent of carbon and had a Brinell hardness of 153; the second (steel *B*) contained 0.44 per cent of carbon and had a Brinell hardness of 129. Both showed the usual ferrite-pearlite structure.

METHODS OF TEST

The bronzes were tested under rolling and sliding friction without lubrication at atmospheric and elevated temperatures up to 350° F. (175° C.); under rolling and sliding friction with lubrication at atmospheric temperatures; and under tension, single-blow impact

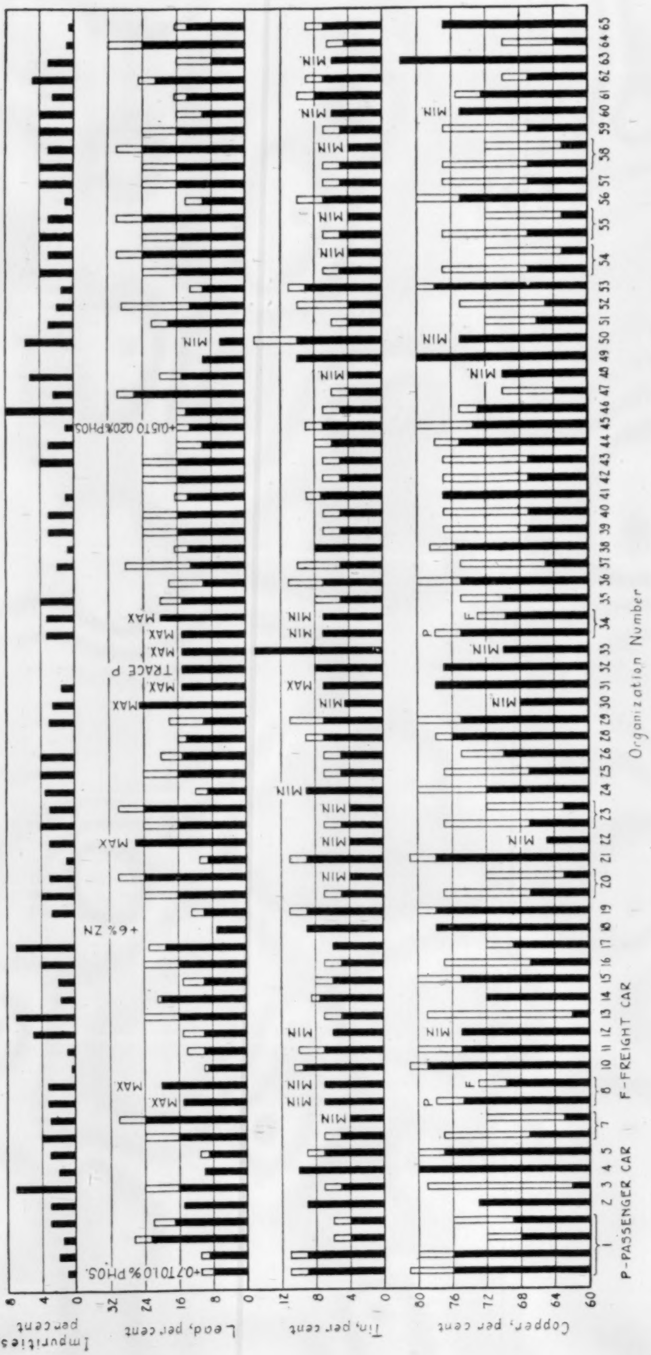
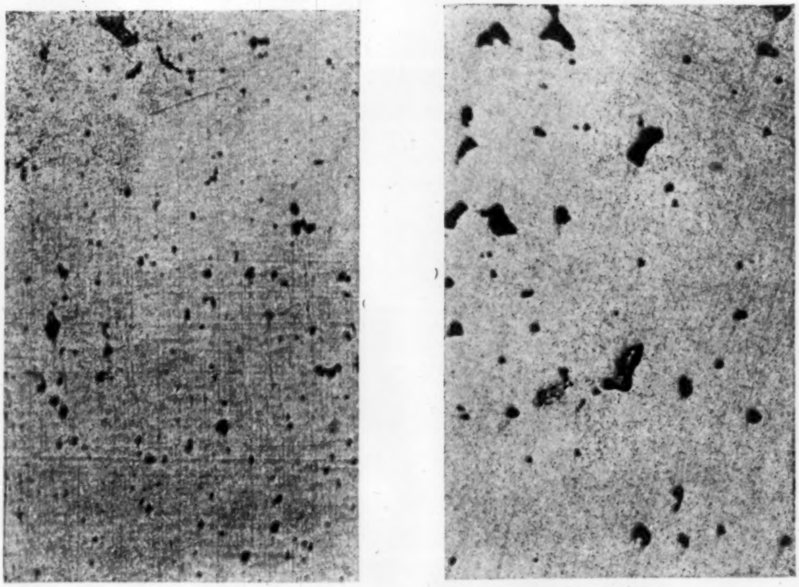


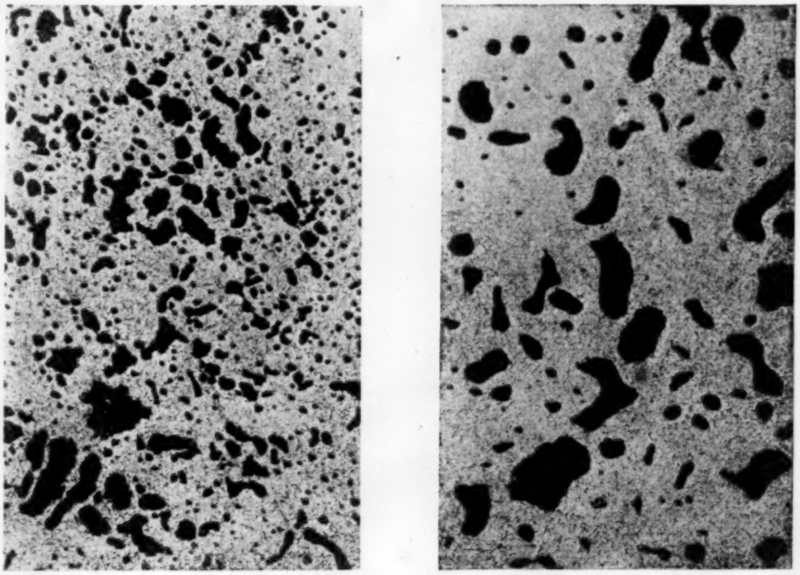
FIG. 1.—Representative Specifications for Journal Bearings Used in the United States During 1925.

Tolerances are indicated by the differences between solid and hollow blocks. Different carriers are represented by number.

Chill Cast Sand Cast



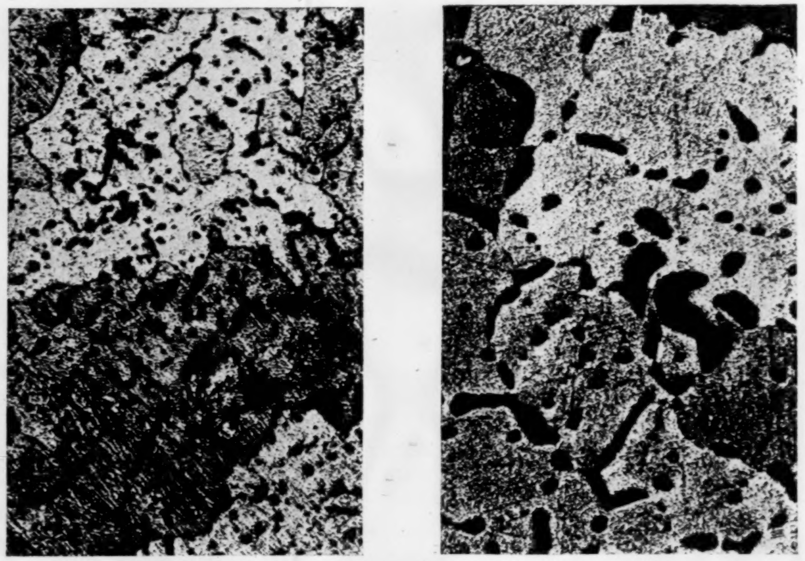
0.25 per cent Lead



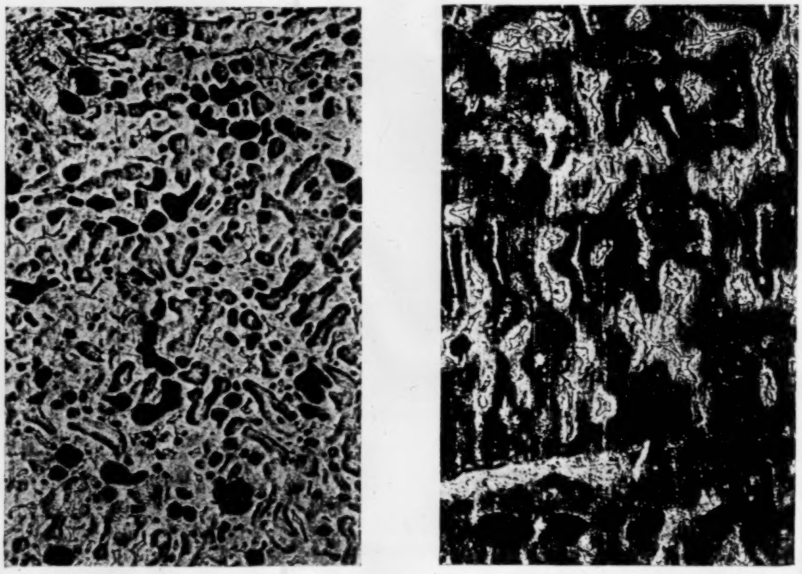
24.0 per cent Lead

FIG. 2.—Photomicrographs of Some of the Unetched Specimens of the Bronzes Varying in Lead (× 100).

Chill Cast Sand Cast



0.7 per cent Tin



10.0 per cent Tin

FIG. 3.—Structure of Some of the Bronzes Varying in Tin. Etched with Solution of 3 Parts NH_4OH plus 1 Part H_2O_2. Then Etched with Acidic Ferric-Chloride Solution (× 100).

(Izod test), and repeated pounding at temperatures from atmospheric to 600° F. (315° C.).

The wear tests were made on an Amsler wear testing machine,[1] illustrated in Fig. 4. The single-blow impact tests were made with an Izod machine with special heating equipment shown in Fig. 5.

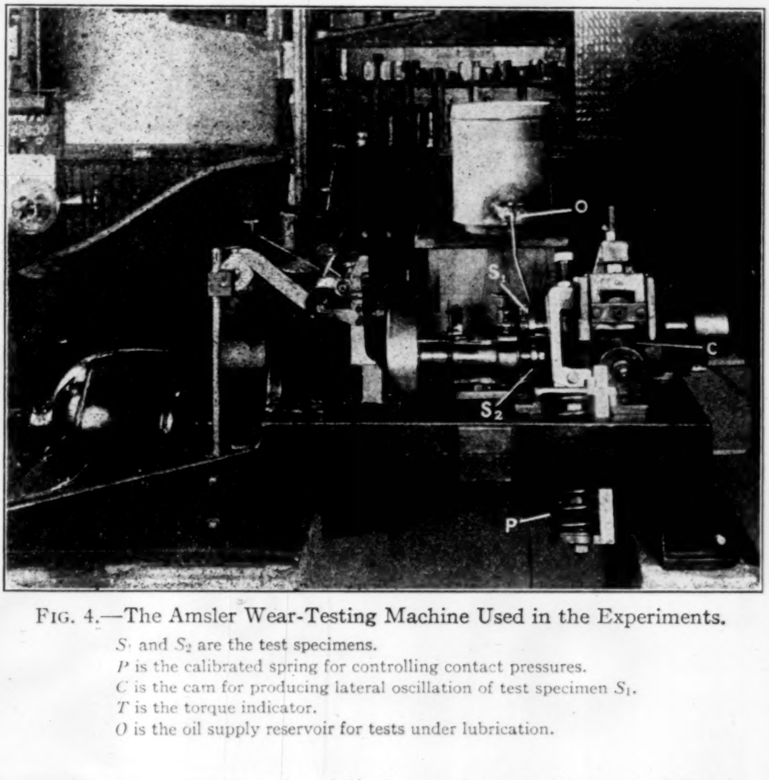


FIG. 4.—The Amsler Wear-Testing Machine Used in the Experiments.

S_1 and S_2 are the test specimens.
P is the calibrated spring for controlling contact pressures.
C is the cam for producing lateral oscillation of test specimen S_1.
T is the torque indicator.
O is the oil supply reservoir for tests under lubrication.

The tension tests were made with the equipment illustrated in Fig. 6,[2] and the pounding tests were made on a testing machine of special construction illustrated in Fig. 7.

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[1] A description of this machine was given by F. P. Hitchcock in "Testing" (Pullman Publishing Co., New York), Vol. 1, p. 147, under the title "A Universal and Practical Machine for Determining the Resistance of Metals to Wear Under Various Kinds of Friction Encountered in Practice." See also Works Equipment, *The Automobile Engineer*, Vol. 17, p 212 (1927).

[2] The Marten extensometer and other equipment used in the tensile tests were described in an earlier paper by the author, H. J. French, "Methods of Test in Relation to Flow in Steels at Various Temperatures," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 7 (1926). See also *Letter Circular 238* of the U. S. Bureau of Standards.

DISCUSSION OF RESULTS

Wear Tests

Since no standardized laboratory technique has been developed for testing bearing metals, it becomes necessary first to apply chosen methods of test to metals of which the performance characteristics are generally well known. Having shown that the results are consistent with performance in service, these methods may then be applied to the study of other alloys. These features are mentioned primarily to answer possible questions concerning the need for extended laboratory tests of metals with generally well-known characteristics in bearing service.

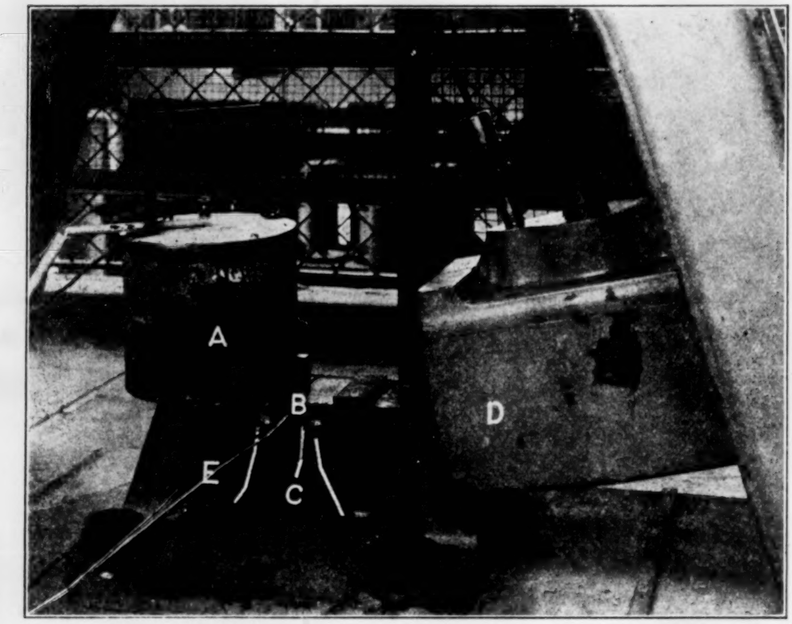

FIG. 5.—Part of the Equipment Used in the Notched-Bar Impact Tests.

A is the equalizing furnace removed just before striking the blow with the tup *D*.
B is the special vise heated with resistance wires *C*.
E is the thermocouple inserted in the test specimen for measuring test temperature.

Probably the chief obstacles in the development of an adequate laboratory technique for bearing metals have been (1) the difficulty of resolving the ordinary service requirements, involving frictional and wearing properties and the resistance to various mechanical stresses, into essential fundamental components, and (2) the difficulty of securing reproducible and interpretable wear test data. The latter was given particular attention in this investigation and it has already been shown[1] that reproducible results can be obtained with the selected methods of test for wear.

The results of the experiments show that no one type of wear test can, by itself, be expected to give adequate information for general comparisons. Wear tests at atmospheric temperatures will not necessarily place alloys or variables of manufacture in the same order as tests at elevated temperatures.

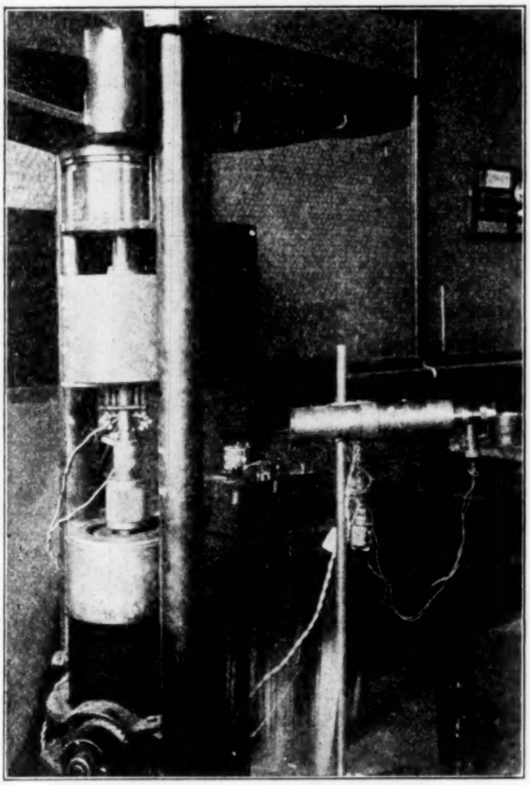

FIG. 6.—Equipment Used in the Tension Tests.

A bearing metal is ordinarily subjected to a complex set of conditions in service and forms but one part of a complicated system. This includes at least the bearing metal itself, the shaft, the lubricant and frequently, in railroad service, non-metallic abrasive particles. Service conditions may cover the range from complete film lubrication through boundary lubrication to conditions in which there is no lubrication at all. The wear tests which were made are near the two extremes and may not indicate what is to be expected under the unstable conditions of boundary lubrication.

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[1] H. J. French, "Wear Testing of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 212 (1927).

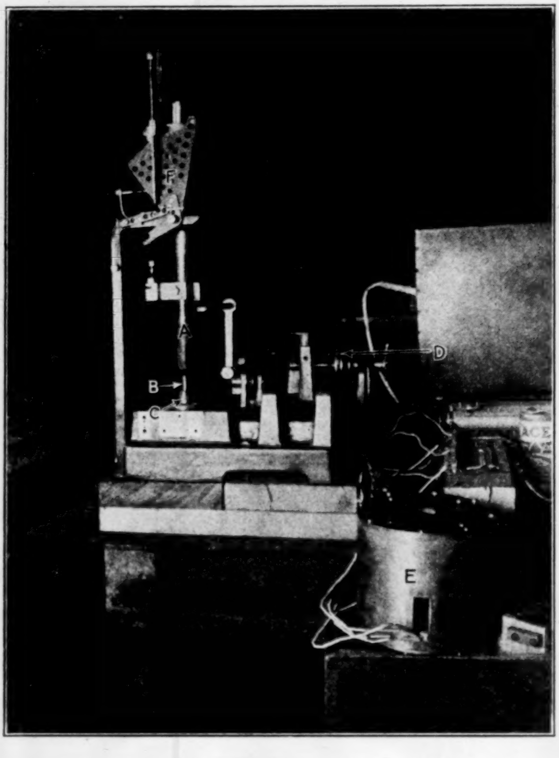


FIG. 7.—Equipment Used in the Pounding Tests.

A is the falling weight. (7.15 lb. through 2 in. in the described tests.)

B is the specimen.

C is the anvil.

D is the operating mechanism of a Stanton repeated-impact testing machine.

E is the furnace within which specimens were tested at different temperatures.

F is the mechanism to insure individual blows free from the auxiliary blows associated with the rebound of the falling weight.

The author is indebted to E. W. Staples, Research Associate for the Bunting Brass and Bronze Co., Toledo, Ohio, for the design and construction of this machine.

It is generally recognized that with complete film lubrication wear is unimportant. It is also probable that difficulties would be encountered in securing reproducible data on wear under conditions of boundary lubrication, so that it seemed logical to turn to unlubricated wear tests which could be carried out without the production of excessive frictional temperatures.

Some engineers are of the opinion that it is impracticable to interpret tests made without lubrication, but until methods of test become available in which reproducible data can be secured and by which it is possible to isolate the characteristics of the metals from the design and fit of the bearings, tests without lubrication can properly be considered to give at least some information of interest and fundamental importance.

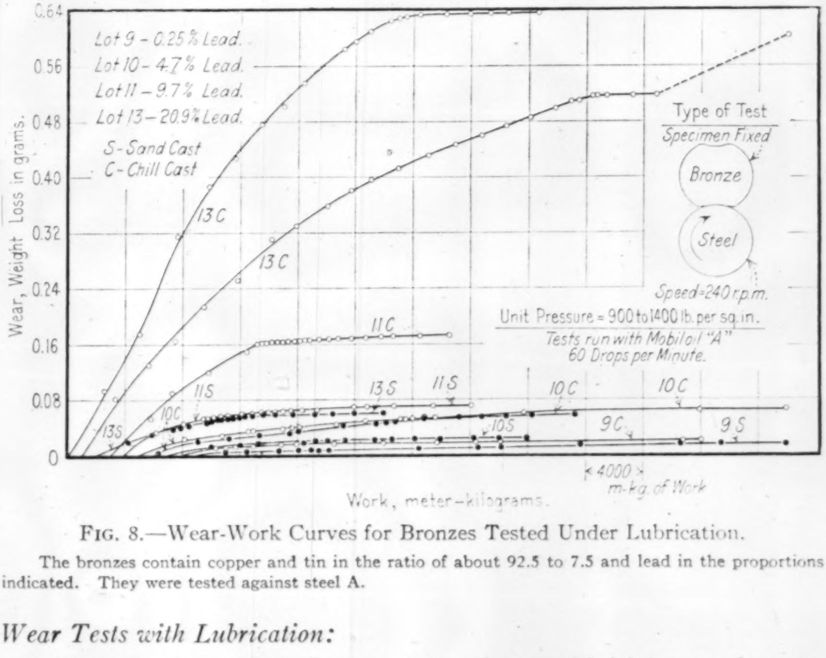


FIG. 8.—Wear-Work Curves for Bronzes Tested Under Lubrication.

The bronzes contain copper and tin in the ratio of about 92.5 to 7.5 and lead in the proportions indicated. They were tested against steel A.

Wear Tests with Lubrication:

Figure 8 shows that the amount and rate of initial wear, in tests under lubrication, varied widely for the different bronzes, but after an initial period, the wear-work curves (the same applies to wear-revolution curves) turned toward the horizontal axis. Thus, after a preliminary "wearing-in" period, the wear rate dropped to very low values in the presence of a lubricant and it became impracticable to develop differences between the different alloys with any degree of certainty. Therefore, it seemed necessary to make tests without lubrication to throw additional light upon the wearing properties of the bronzes.

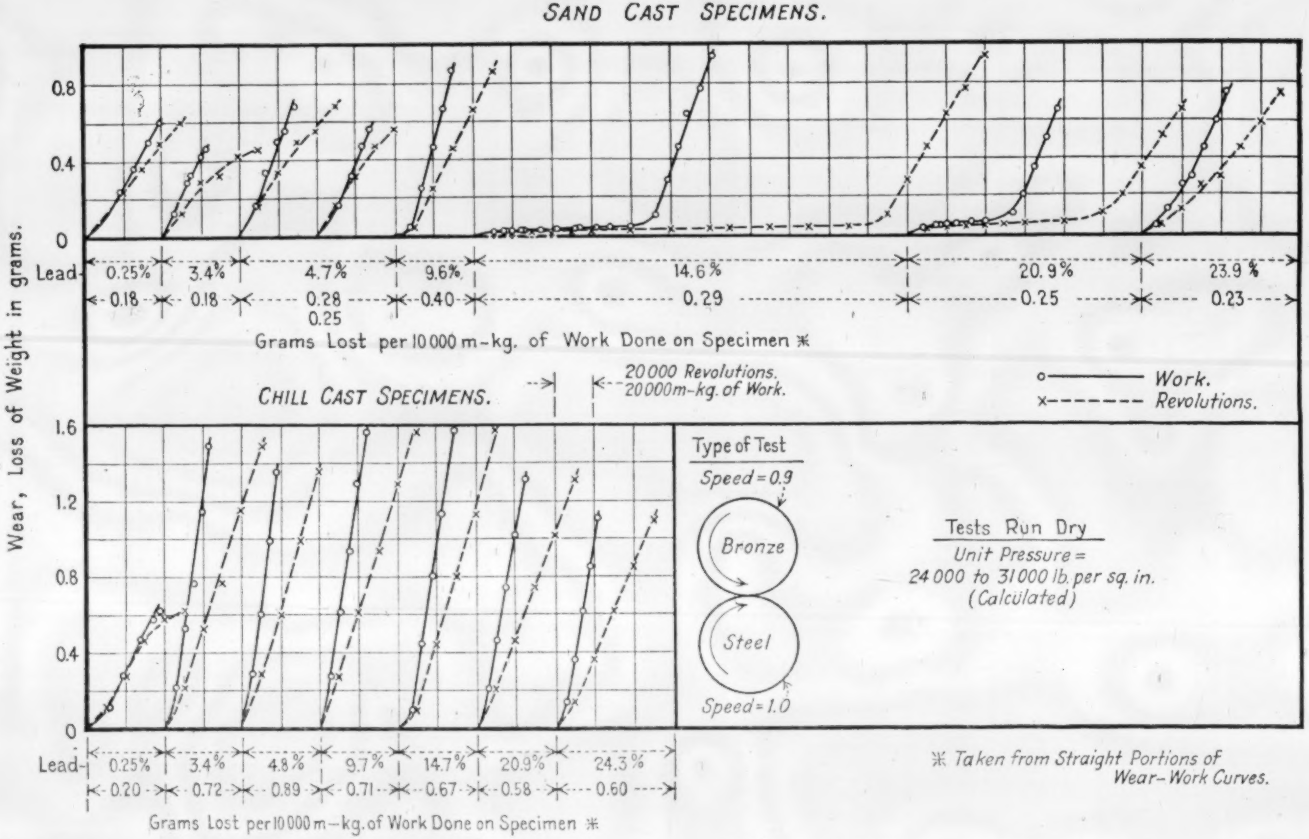


FIG. 9.—Wear-Revolution and Wear-Work Curves for Bronzes Tested Without Lubrication in the Amsler Wear-Testing Machine.

The bronzes contain copper and tin in the ratio of about 92.5 to 7.5 and lead in the proportions indicated. They were tested against steel A.

It should be recognized that the magnitude and duration of the "wearing-in" period will be quite largely dependent upon the lubricant and the relation of this lubricant to the initial condition of the surface of the bearing metal. They will likewise be dependent upon the accuracy of the alignment of the test specimens in the Amsler wear testing machine. Under such conditions, it is conceivable that the differences in duration and weight losses of the initial period, shown in Fig. 8, may be reduced or obliterated if sufficiently smooth contact surfaces are provided and a sufficiently close alignment of the test specimens is obtained.

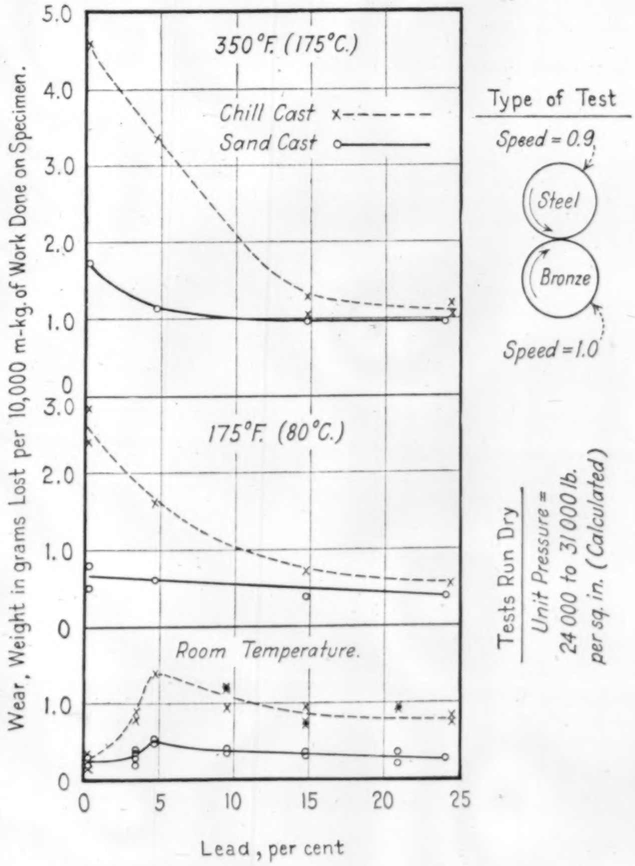


FIG. 10.—Effect of Lead on the Wear Per Unit of Work for Bronzes Subjected to Friction Without Lubricants at Different Temperatures.

The bronzes contain copper and tin in the ratio of about 92.5 to 7.5. They were tested against steel A.

Wear Tests Without Lubrication:

Typical wear-work and wear-revolution curves for tests without lubrication are shown in Fig. 9. Most of these show a "wearing-in" period but this is not so marked as in the tests made with lubrication (Fig. 8). Instead of approaching the horizontal axis after a preliminary period as in the tests under lubrication, they assume a well-defined slope representing a relatively high rate of wear. So long as no unusual circumstances arise to modify the conditions of test, the

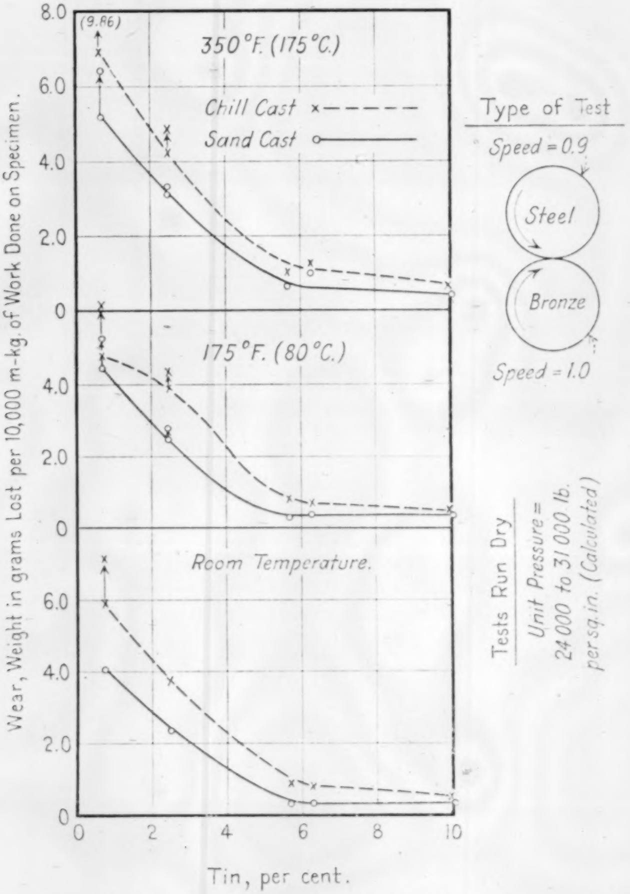
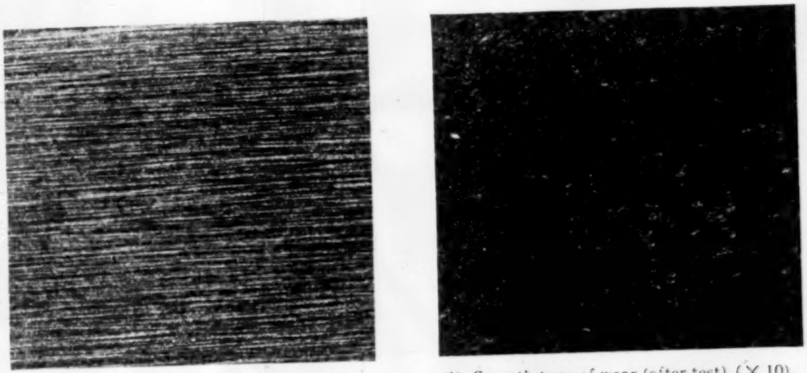


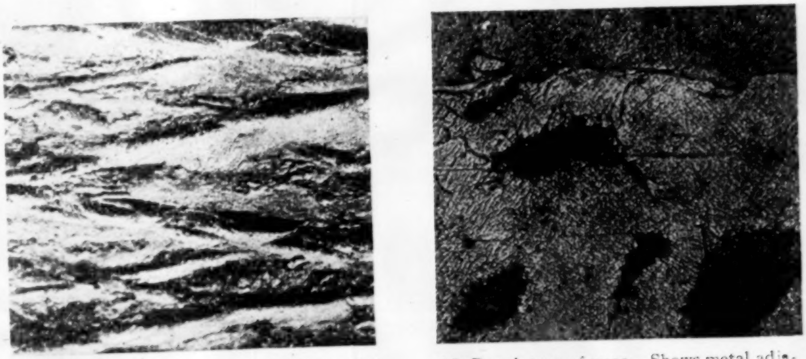
FIG. 11.—Effect of Tin on the Wear Per Unit of Work for Bronzes Subjected to Friction Without Lubricants at Different Temperatures.

The bronzes contain copper and lead in the ratio of about 84 to 16. They were tested against steel A, at atmospheric temperatures and against steel B at other temperatures.



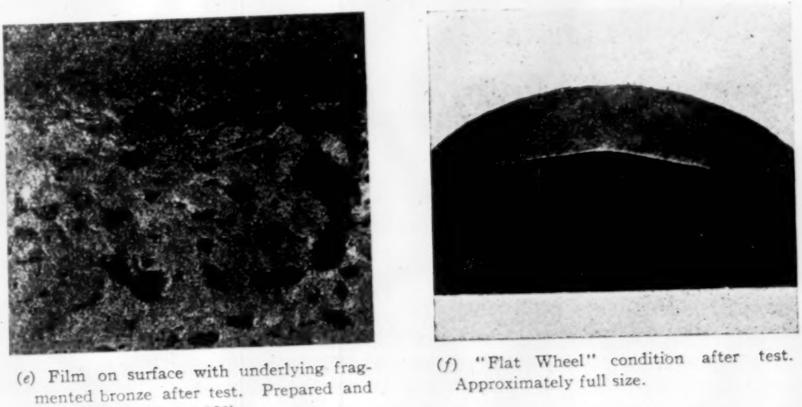
(*a*) Original machined surface (× 10).

(*b*) Smooth type of wear (after test) (× 10).



(*c*) Very rough with ridges (after test) (× 10).

(*d*) Rough type of wear. Shows metal adjacent to the surface after test. Etched with solution of 3 parts NH_4OH plus 1 part H_2O_2 (× 250).



(*e*) Film on surface with underlying fragmented bronze after test. Prepared and etched as for (*d*) (× 250).

(*f*) "Flat Wheel" condition after test. Approximately full size.

FIG. 12.—Surfaces of Some of the Wear Test Specimens.

slopes of the wear-work or wear-revolution curves may be assumed to represent a more or less definite state of equilibrium and may be used for comparison of the different bronzes when subjected to friction in the absence of lubricants. Such comparisons are included in Figs. 10 and 11.

Wear of Chill versus Sand Castings:

In general, the alloys cast in metal molds wore more rapidly than corresponding alloys cast in sand molds. This seems to be of fundamental importance as well as of practical interest. In fact, in one or two cases the reduction in the diameter of the test specimens to bring the wearing surface further below the zone of maximum chill resulted in an appreciable reduction of the wear rates which then approached those of the corresponding sand-cast alloys. On the other hand, the sand-cast bronzes usually caused more wear on the steels with which they were in contact than corresponding alloys cast in metal molds. However, these weight losses were all small in comparison with those of the bronzes.

Influence of Chemical Composition on Wearing Properties:

Consideration of the bronzes on the basis of chemical composition shows three groups of alloys which have distinctly different wearing properties in either the sand-cast or chill-cast conditions. These three are (1) the copper-lead mixture with small proportions of tin; (2) the bronze with 0.25 per cent of lead which contains about 92.5 per cent of copper and 7.5 per cent of tin; and (3) the high-lead bronzes containing about 12 to 25 per cent of lead, 5 to 10 per cent of tin, and the balance copper.

The copper-lead mixture, with small proportions of tin, showed good anti-frictional properties, as indicated by low torque in the wear tests without lubrication, but had a very high wear rate at all test temperatures and quickly assumed a rough surface and a "flat-wheel" condition illustrated in Fig. 12 (*f*). It does not appear to be suited for general railroad bearing service.

The bronze with 0.25 per cent of lead showed intermediate to high rates of wear, except at atmospheric temperatures. It had poor anti-frictional properties, quickly assumed an exceedingly rough surface (Figs. 12 (*c*) and (*d*)) and rapidly developed the "flat-wheel" condition in the absence of a lubricant. It likewise caused relatively high wear on the axle steel with which it was in contact. However, it had a small "wearing-in" period, including low weight loss in tests under lubrication and appeared best suited for applications in which lubrication and operating temperatures close to atmospheric can be maintained.

The third group, comprising the high-lead bronzes with 12 to 25 per cent of lead and 5 to 10 per cent of tin, had low to intermediate wear rates and favorable anti-frictional properties in tests without lubricants, particularly in the sand-cast condition. They also produced relatively little wear on the steels with which they were in contact. The worn surfaces of these bronzes were relatively smooth (Fig. 12 (*b*)), and the sand-cast specimens showed a marked tendency to form films (Fig. 12 (*e*)) which usually reduced the torque and wear.

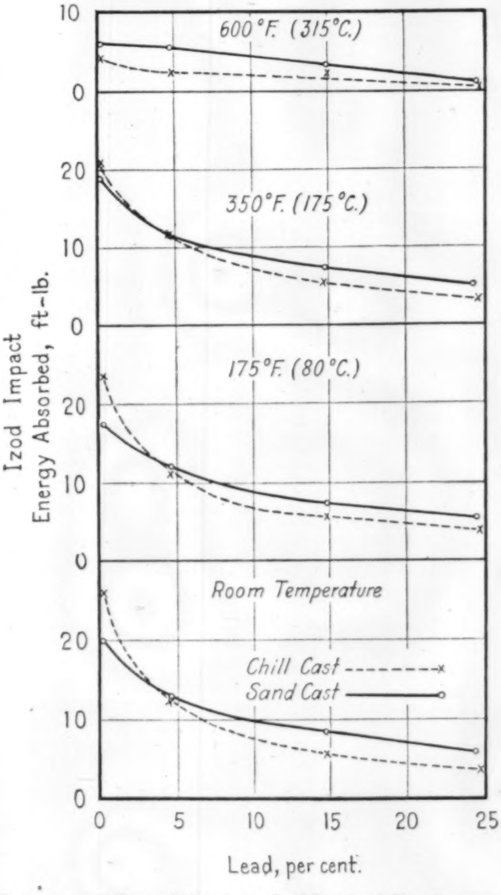


FIG. 13.—Effect of Lead on the Notched-Bar Impact Resistance of Bronzes at Different Temperatures.

The bronzes contain copper and tin in the ratio of about 92.5 to 7.5.

In the tests made in the presence of a lubricant, these alloys showed a relatively high weight loss in the "wearing-in" period, which was also of long duration. Such bronzes appear best suited for service in which complete lubrication cannot be guaranteed and where high frictional temperatures may, therefore, be expected. However, their anti-frictional properties in the tests under lubrication were somewhat superior to the anti-frictional properties of the bronze with 0.25 per cent of lead, which showed more erratic torque values. Thus, the high-lead bronzes also may have one advantage over the low-lead bronze for service at ordinary temperatures under lubrication.

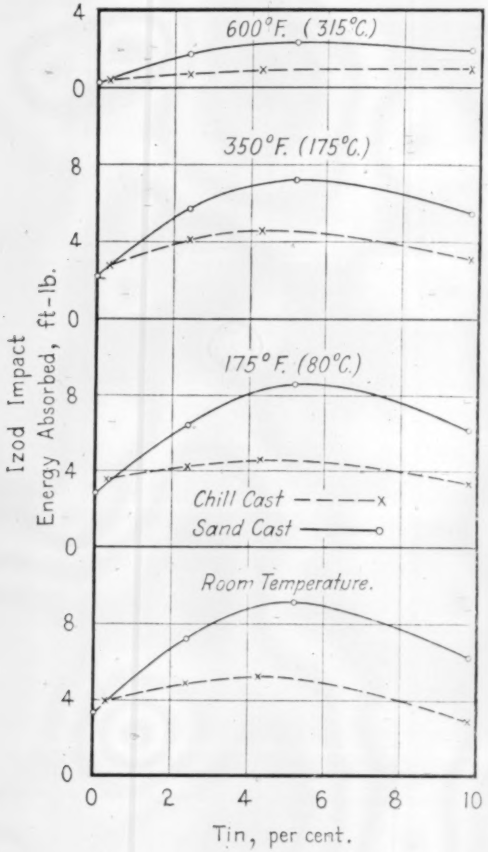


FIG. 14.—Effect of Tin on the Notched-Bar Impact Resistance of Bronzes at Different Temperatures.

The bronzes contain copper and lead in the ratio of about 84 to 16.

This brief and general discussion of the results of the wear tests will serve to emphasize the fact that no one of the alloys tested is best suited for all purposes and that each group has certain outstanding characteristics from the viewpoint of wearing properties. However, information is needed concerning other important properties before a logical choice may be made.

Mechanical Tests

Impact Tests:

The results of the Izod impact tests are summarized graphically in Figs. 13 and 14 and show the following five important features:

1. Bronzes with small amounts of lead show better resistance to notched-bar impact than the high-lead bronzes. Hence, lead may be said to exert a deleterious effect on the notch-toughness of bronzes.

2. Increase of tin in copper within the alpha solid solution range tends to increase impact resistance of bronzes; much more than this amount of tin, which results in the formation of appreciable amounts of the alpha-delta eutectoid, tends to reduce the notch-toughness.

3. In the high-lead bronzes containing from about 10 to 25 per cent of lead and from 2 to 10 per cent of tin the sand-cast metal has a generally better notch-toughness than corresponding chill-cast metal. This is probably due to the form and distribution of the lead particles, which are smaller and more than counteract the beneficial effects of the fine-grained matrix in the chill castings.

4. In bronzes with small amounts of lead, this order of superiority may be reversed. In such alloys, the full benefits are derived from the fine-grained structure and the chill-cast bronzes may show higher notch-toughness than the corresponding sand-cast bronzes with a coarse-grained structure. With intermediate proportions of lead, around 5 per cent, the impact resistance is about the same for the sand-cast and chill-cast metals. Here the benefits of fine grain in the chill castings seem to be balanced by the deleterious effects of the finely disseminated lead.

5. Increase in temperature to 600° F. (315° C.) tends to reduce the differences in impact resistance referred to in items 1 to 4 inclusive, and tends to lower the notch-toughness. However, the decrease in impact resistance of the bronzes is more marked between 350 and 600° F. (175 and 315° C.) than between atmospheric temperatures and 350° F. (175° C.).

Tension Tests:

The results of the tension tests are summarized in Figs. 15 and 16.

Increase in lead from 0.25 per cent to around 25 per cent produced a general decrease both in the strength factors and ductility at atmos-

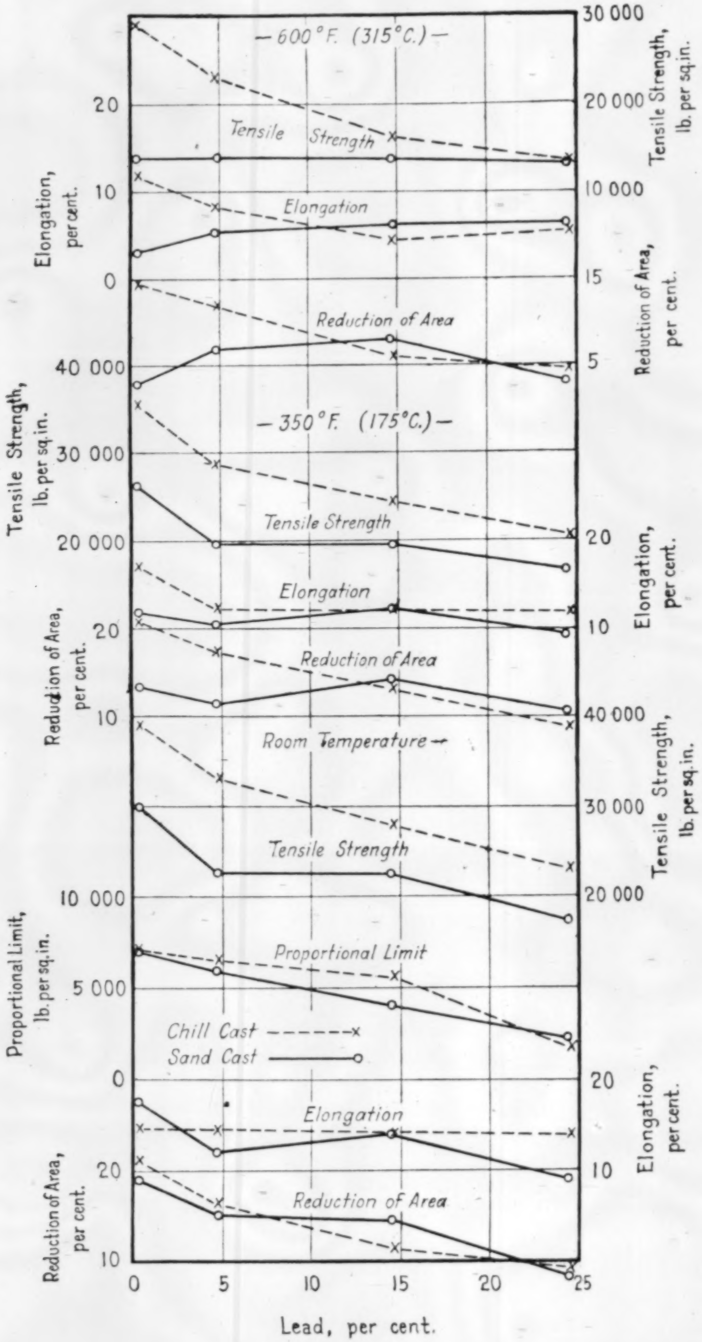


FIG. 15.—Effect of Lead on the Tensile Properties of Bronzes at Different Temperatures.

The bronzes contain copper and tin in the ratio of about 92.5 to 7.5.

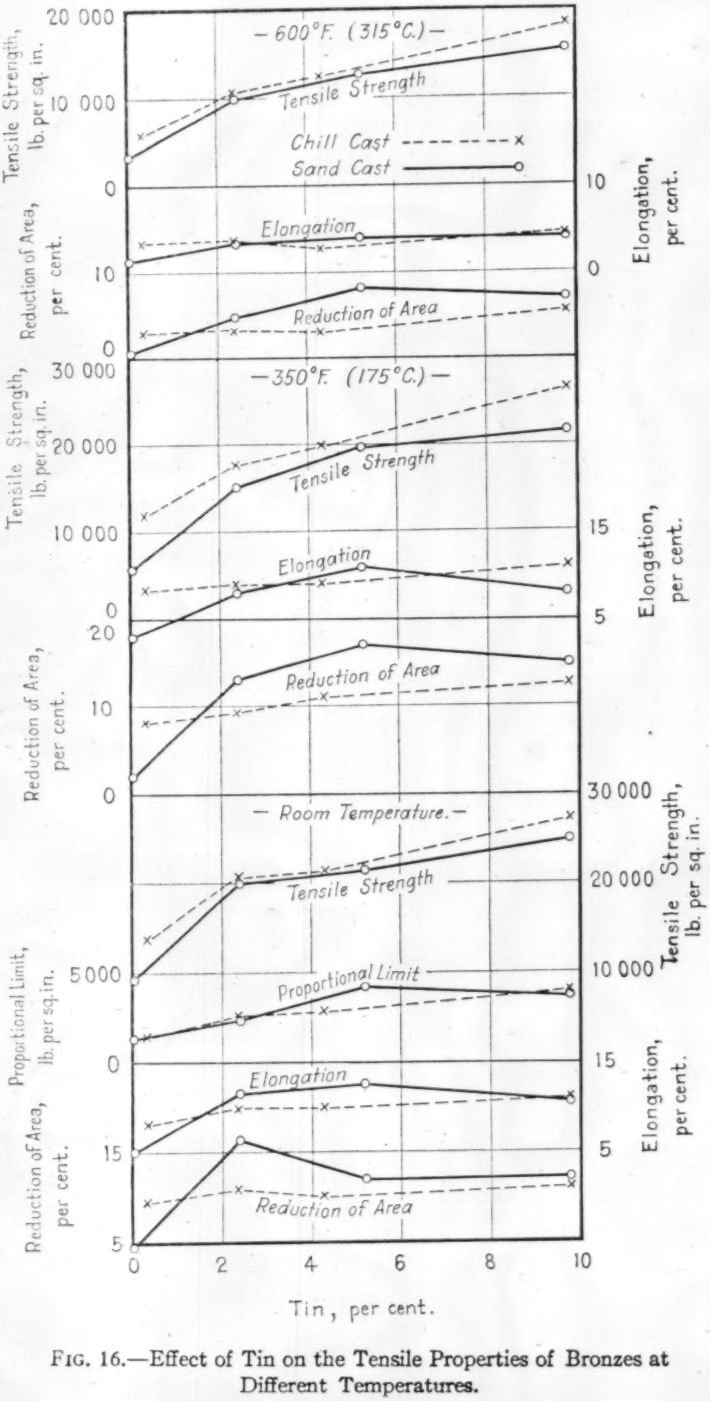


FIG. 16.—Effect of Tin on the Tensile Properties of Bronzes at Different Temperatures.

The bronzes contain copper and lead in the ratio of about 84 to 16.

pheric temperatures. On the other hand, increase in tin from a trace to about 10 per cent resulted in a general increase in the strength factors and ductility. However, this improvement in tensile properties was much larger for the increase between a trace and 2.5 per cent of tin than between 2.5 and 10 per cent.

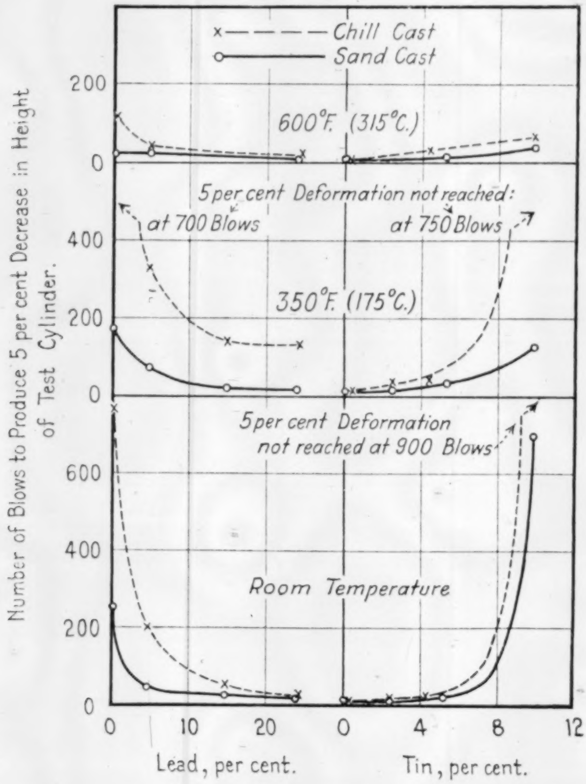


FIG. 17.—Resistance to Pounding of the Different Bronzes at Different Temperatures.

The chill castings quite generally showed a higher tensile strength than the corresponding sand castings and this was frequently, but not always, accompanied by decreased ductility.

These general trends are evident at all test temperatures between 70 and 600° F. (20 and 315° C.), although the magnitude of the differences depends upon the alloys and the temperatures considered. Increase of temperature above atmospheric produced a decrease in the tensile strength of the bronzes, but these changes became smaller as the tin content of the bronzes increased. They were likewise less in a majority of the chill-cast bronzes than in the corresponding sand-cast bronzes, but did not seem to be affected in any regular manner by variations in the proportions of lead.

Pounding Tests:

Comparisons are given in Fig. 17 of the number of blows required to produce a 5-per-cent decrease in the height of test cylinders which originally had a length of 0.984 in. (25 mm.) and a diameter of 0.394 in. (10 mm.). Here the number of blows may be taken to represent the resistance to pounding.

Increase in lead from 0.25 to around 5 per cent produced a marked decrease in the resistance to pounding; with lead increase from around 5 to 25 per cent a further but small decrease was observed.

The resistance to pounding increased slightly with tin increase from a trace to about 5 per cent and showed a very large increase when the tin was raised from 5 to 10 per cent. In practically all cases the chill-cast bronzes showed higher resistance to pounding than the corresponding sand-cast bronzes.

The described effects from variations in chemical composition and method of casting were similar at 350 and 600° F. (175 and 315° C.) to those at atmospheric temperatures, although the magnitude of these effects decreased with increase in temperature, particularly between 350 and 600° F. (175 and 315° C.). Many of the bronzes showed higher resistance to deformation in both the tension tests and pounding tests at 350° F. (175° C.) than they did at atmospheric temperatures.

COMPARISONS BETWEEN EXPERIENCES IN SERVICE AND THE RESULTS OF THE EXPERIMENTS

The tests described in this report do not characterize completely the properties which are of interest in the application of bronzes in bearings. Nevertheless, certain general trends were observed which can be considered in the light of practical experience in railroad work.

The sand-cast bronzes quite generally showed lower wear than the corresponding chill-cast bronzes. At first glance this may seem contradictory to the recent experiences of some railroads which have found their overall costs for locomotive bearings to be lower with chill-cast bronzes than with sand-cast bronzes. It is not known definitely whether or not these practical comparisons are based on alloys of similar chemical composition, but a sufficient number of such reports have been encountered to justify consideration of the relations between such experiences in service and the described results of laboratory tests.

Under ordinary conditions there is a difference in machining properties in favor of chill-cast metal. Easy machining should not only contribute directly to lower costs, but also indirectly through increased bearing life associated with the better fits which would be a natural consequence of good working properties.

Another matter for consideration is the character of the failure of locomotive bearings. It has been common practice to state that bearings have worn out when they are no longer serviceable, whether the unserviceability was the result of abrasion, fracture under impact at high temperatures, so-called pounding out, or other causes.

The experience of one railroad has been that "chill brass is largely removed from wear whereas the sand-cast brass breaks up more frequently in service."[1] This is consistent with the results of the laboratory experiments, which showed that the chill-cast bronzes wore more rapidly than corresponding sand-cast bronzes, but raises the question of why the sand castings break up more frequently in service since they showed better notch-toughness than the chill castings.

Such breakage rarely takes place under the first application of shock in service except in imperfect castings. It is the result of repeated shocks which can be expected to work-harden and deform the bronze. As shown in the laboratory pounding tests (Fig. 17), the chill-cast bronzes had a higher resistance to deformation than corresponding sand-cast bronzes. Since the sand castings deform more readily there would be a greater tendency than with chill-cast metal for the bearing to "pound out" under the repeated shocks in service. This would have the effect of increasing the shock intensity and possibly account for the fact that the sand-cast bearings more often break up in service than do the chill-cast bearings.

There is perhaps room for considerable argument on the question of the superiority of one metal or method of casting over another in respect to the effects of pounding in service. Very few railroad castings are applied without grease grooves of some sort. Service stresses are concentrated at these grooves and result in conditions comparable to those obtaining in the impact tests, although in service the stresses are repeatedly applied as in the pounding tests. Certainly a more complete picture is obtained from the results of both the pounding tests and impact tests than is given by either test alone.

While the foregoing comparisons indicate a closer correlation between some service experiences and the pounding tests, one manufacturer of bearing metals has cited a case where a closer correlation appears to exist between practical performance and the impact tests. His comments[1] are as follows:

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[1] Private communication.

> For example, we have the early failures of poured-on lateral plates used for facing the driving truck and trailer boxes adjacent to the hub of the wheel. These lateral plates cause so much trouble by failure from excessive wear and cracking that many of our railroad customers have been replacing the poured-on lateral plate with sand castings applied to the box by means of studs. The poured-on lateral plate is applied at the railroad shops. Molten bronze is poured on the box face after the bronze bearing has been pressed into the box. The lateral is held against the steel box face by means of dovetailing. The lateral plate is then machined and lubrication grooves cut in the brass. The poured-on lateral plate, due to its rapid cooling against the steel face of box, has a very close fine grain and is very brittle in structure. It is a chill casting in every sense of the word.
>
> Sand-cast lateral plates, however, do not show the failures characteristic of the poured-on lateral plates and are rapidly replacing the old style plate.

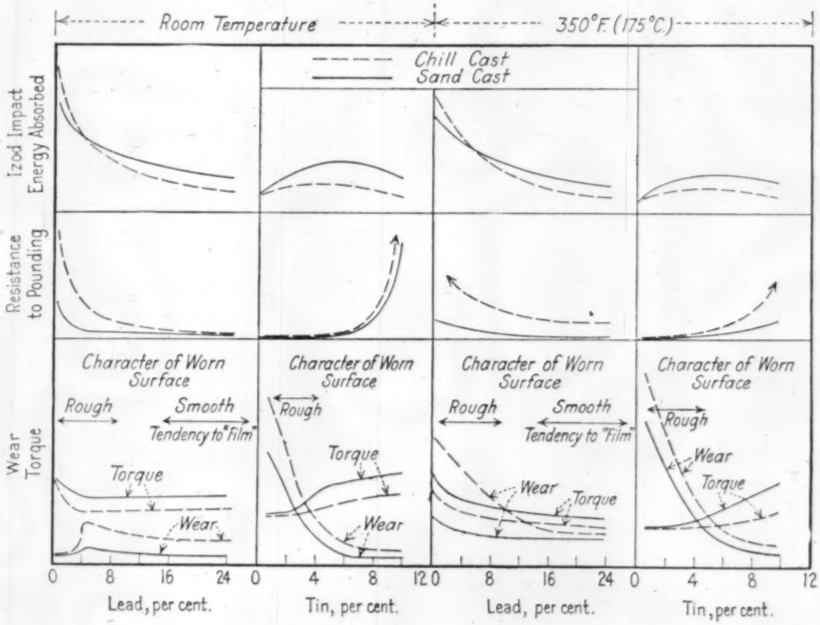

FIG. 18.—Graphical Summary of Important Features Shown in the Wear Tests and Mechanical Tests.

Values are not given for the ordinates, as this figure is intended only for general comparisons. Details are included in preceding figures.

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[1] Private communication.

Each application should be given special consideration and tests chosen which will most closely simulate the conditions of service. It is not only necessary to consider the service conditions but also to consider carefully the interpretation of the results obtained from the selected types of test.

APPLICATION OF THE DIFFERENT BRONZES

A graphical summary of the important features shown in both the wear tests and the mechanical tests is given in Fig. 18 for use with the following discussion.

While the bronzes studied fall within a limited range of chemical composition, there are alloys in the group which should meet a variety of service conditions. Brief consideration will be given to their fields of application as judged from the described laboratory tests, but it should not be inferred that any one alloy is the best for a given purpose, since a narrow range of alloys is considered and subsequent comments are primarily concerned with the interpretation of the laboratory test data.

Bronzes with less than about 5 per cent of tin do not appear to be suited for bearing service since their wearing properties and mechanical properties are both relatively poor (Fig. 18). They will, therefore, be excluded from further consideration.

There are decided advantages in the high-lead bronzes for service where lubrication cannot be maintained. These bronzes, containing about 12 to 25 per cent of lead, did not wear as rapidly as the low-lead bronzes nor assume as rough a surface in tests without a lubricant. Provided wear is the chief factor to be considered and an approximately constant copper-tin ratio is maintained, variations in lead seem unimportant within the range 12 to 25 per cent (Fig. 18).

Where it is also important to have high resistance to pounding, high notch-toughness and static strength at atmospheric and elevated temperatures, lead may advantageously be kept near the low limit of the specified range, since the strength and notch-toughness decrease with increase in the lead. However, these differences are relatively small and for many purposes the bronzes containing from, say, 15 to 25 per cent of lead may be considered to be interchangeable.

When the mechanical properties become of primary importance and lubrication can be maintained so that wear is of secondary interest, there are advantages in low-lead and high-tin (Fig. 18). Both favor high strength and resistance to pounding of the bronzes. Low lead and increase in tin up to the point where appreciable proportions of the eutectoid was observed, likewise improved the notch-toughness.

Furthermore, the high-tin bronzes with appreciable proportions of lead, and the low-lead bronzes with about 5 to 8 per cent of tin, maintained a superiority over the high-lead bronzes in their mechanical properties at elevated temperatures, although this was not of the same magnitude as at atmospheric temperatures.

SPECIFICATIONS FOR RAILROAD BEARING BRONZES

While variations in the proportions of lead and tin in the bronzes are important in relation to the requirements of service, the effects of some of these changes in chemical composition are much less important than variations arising from the method of casting. For example, the differences in wearing properties produced by increase in lead from 15 to 25 per cent are smaller than those resulting from a change from sand to chill molds. Other examples will be found by examination of the results of the different tests summarized graphically in Fig. 18.

On the whole, there does not seem to be any real justification for the very wide variation in the specifications of different carriers for bearings of similar design subjected to similar service, as outlined in the introduction to this paper. It will be seen from Fig. 1 that there are not only wide differences in the type compositions employed for similar service but also what appear, in the light of the described tests, to be unnecessary restrictions in the prescribed limits of chemical composition.

The fact that car journal bearings are replaced without serious difficulties on foreign lines with metals varying widely in chemical composition, confirms the viewpoint reached from the laboratory tests that the existing specifications may to advantage be revised and brought more nearly into conformity. Some of the wide variations appear unimportant, and unnecessary restrictions are now encountered. Likewise, a generally improved performance could probably be obtained by more uniform specifications based on the selection of the type compositions best adapted to certain broad requirements.

The effects of impurities upon copper-tin-lead alloys were not considered in this report. Before complete specifications can be drawn, information should be obtained on the effects produced by impurities, such as iron and antimony, and the effects of elements intentionally added, such, for example, as phosphorus, zinc, nickel, etc. Information on the effect of annealing or other methods of control of the amount of "coring" of the alpha solid solution would also be pertinent.

However, it is of interest to note that the grouping of the bronzes resulting from the described laboratory tests is consistent with the limits of tin and lead in the Society's Tentative Specifications for Car and Tender Journal Bearings, Lined (B 67 - 27 T).[1] In these specifications tin may vary from 5 to 7 per cent and lead from 15 to 22 per cent.

SUMMARY

A study was made of the wearing and mechanical properties of two groups of copper-tin-lead alloys in both chill-cast and sand-cast conditions. The first group consisted of bronzes with a copper-tin ratio of about 92.5 to 7.5 and lead varying from 0.25 to 25 per cent; in the second group the bronzes contained a copper-lead ratio of about 84 to 16 and tin varying from a trace to 10 per cent.

The bronzes were tested under rolling and sliding friction without lubrication at atmospheric and elevated temperatures up to 350° F. (175° C.); under rolling and sliding friction in the presence of a lubricant at atmospheric temperatures; and under tension, single-blow impact and repeated pounding at temperatures from 70 to 600° F. (20 to 315° C.). The results of these tests developed the following features:

1. No one of the selected laboratory tests yielded information which was, by itself, adequate for general comparisons. Each of the wear tests and the mechanical tests contributed information of value in developing the characteristics of the different bronzes.

2. Variations in the chemical composition of the bronzes produced major changes in properties, but within certain ranges variations in chemical composition were much less important than variations in the method of casting.

3. As a general rule, chill-cast bronzes wore faster and had lower notch-toughness than corresponding sand-cast bronzes, but showed better resistance to pounding and higher tensile strength at temperatures between 70 and 600° F. (20 and 315° C.). The few exceptions to this are shown in the body of the report.

4. In alloys with a practically constant ratio of copper to tin, increase in lead produced a general improvement in wearing properties, but this was more marked between 0.25 and 12 per cent than between 12 and 25 per cent of lead. At the same time, the resistance to pounding, notch-toughness and tensile strength decreased.

5. In alloys with a practically constant ratio of copper to lead, increase in tin from 0.7 to around 5 per cent resulted in a marked

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[1] *Proceedings*, Am. Soc. Testing Mats. Vol. 27, Part I, p. 721 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 128.

decrease in the wear in the tests without a lubricant and a general improvement in the wearing properties. Further increase in tin, from around 5 to 10 per cent, did not materially modify the wearing properties but resulted in improved tensile strength and resistance to pounding. There was also an improvement in notch-toughness with increase in tin to about 7 per cent representing the proportion above which appreciable amounts of the brittle alpha-delta eutectoid appeared. With higher tin a slight decrease was observed in the impact values.

6. The bronzes with less than about 5 per cent of tin did not seem to have the combination of mechanical and wearing properties necessary for good service as bearings. Where wearing properties are of primary importance the high-lead bronzes had distinct advantages over the low-lead bronzes. Likewise, the bronzes with around 8 to 10 per cent of tin (and 12 to 15 per cent of lead) seemed better adapted to service where the chief requirements are resistance to pounding and static stress than bronzes with around 4 to 6 per cent of tin.

7. There is a considerable range of chemical composition within which the variations in mechanical and wearing properties are small. This includes a range from 15 to 25 per cent of lead and from around 4 to 7 per cent of tin. Since alloys within the entire range have given reasonably good performance in locomotive and journal bearings, there does not seem to be justification for the wide variation in specifications now used by different carriers for parts subjected to similar service. This refers not only to the type compositions but also to the limits of chemical composition for any one type.

8. It has been shown that reproducible wear tests can be obtained in the laboratory. The results, when combined with suitable mechanical tests, gave comparisons which were consistent with experiences in practical service.

[For Discussion on Wear Testing of Metals, see page 348.—ED.]

WEARING TESTS OF TWELVE-PER-CENT MANGANESE STEEL

By John Howe Hall[1]

Synopsis

The paper describes the development of a test for the wearing properties of 12-per-cent manganese steel that will give results comparable with service experience. A number of crude methods were tried out and abandoned, usually because they were too slow, too clumsy, or were not a true indication of the wearing properties.

It was finally concluded to adopt the method of crushing known amounts of hard stone in a small laboratory stone crusher the jaws of which were made of the steels to be tested. The loss in weight of the jaws was determined at regular intervals. Although recently wear tests have been made in the Amsler and Brinell abrasion machines, it was found that these machines did not simulate the particular kind of wear that manganese steel successfully withstands in service, that is, abrasion associated with severe cold-working such as obtains with the pressure exerted by the crushing of hard material. Recourse was again had to the tedious but reliable jaw crusher.

The paper gives details of tests in such a crusher of manganese steel, plain carbon steel, nickel steel, chromium steel and nickel-chrome steel. Manganese outwore carbon steel about 10 to 1; chromium decreased the wear of the pearlitic steels little if any; nickel decreased the wear to a marked extent.

A little over twenty years ago the company with which the author is associated, of which the late Dr. Henry M. Howe was then Vice-President and Consulting Metallurgist, was confronted with the necessity of developing a test for the wearing properties of 12-per-cent manganese steel that would give results comparable with those obtained in actual service. In an effort to find a method of testing that would not consume too much time, several crude machines were tried out, with results as follows:

1. Bars rubbed back and forth on steel plate, with emery and oil between, under constant pressure. Abandoned as too slow.

2. Bars rubbed on revolving chilled-iron wheel, with emery and oil between, under constant pressure. Abandoned as too slow.

3. Bars rubbed back and forth on sandstone under constant pressure. Abandoned as too clumsy. Manganese steel and carbon steels wore at approximately the same rates.

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[1] Metallurgical Engineer, Taylor-Wharton Iron and Steel Co., High Bridge, N. J.

4. Dragging test bars clamped in shoes under the runners of a "stone-boat," over three miles of sand road. Experience proved that the only possible "load" was a "live" one, consisting of two men, who had a decidedly lively trip. The bars were arranged as shown in Fig. 1.

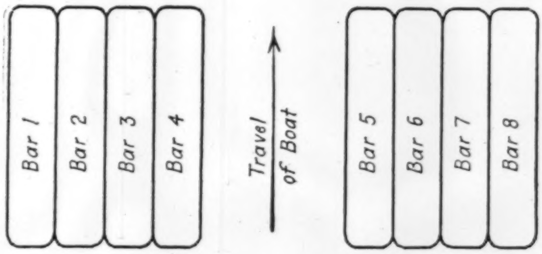

FIG. 1.—Arrangement of Test Bars in "Stone-Boat" Test.

The losses in weight were as follows:

BAR	KIND OF STEEL	LOSS IN WEIGHT, GRAMS
No. 1	0.38 per cent Carbon	20
No. 2	12.00 per cent Manganese	20
No. 3	0.38 per cent Carbon	10
No. 4	12.00 per cent Manganese	10
No. 5	12.00 per cent Manganese	19
No. 6	0.24 per cent Carbon	12
No. 7	12.00 per cent Manganese	9
No. 8	0.24 per cent Carbon	15

KIND OF STEEL	TOTAL LOSS IN WEIGHT, GRAMS LEFT RUNNER	 RIGHT RUNNER
12.00 per cent Manganese	30	28
0.38 per cent Carbon	30	..
0.24 per cent Carbon	..	27

5. Sand blasting. Pieces of approximately equal areas, exposed to a stream of sand for five minutes, lost the following amounts:

KIND OF STEEL	LOSS IN WEIGHT, GRAMS
0.38 per cent Carbon	4.8
0.24 per cent Carbon	3.7
12.00 per cent Manganese	4.2
12.00 per cent Manganese	3.2

It will be noted that in rubbing on sandstone and in dragging over a road, manganese steel bars lost approximately as much weight as carbon steel bars. This was due in part to the obvious fact that when bars of different kinds of steels were tested simultaneously in the same holder, neither the stone nor the road could "follow up" the more rapidly wearing steels perfectly, so that they were to some extent protected by the contiguous bars of steel that wore more slowly. This, however, accounted only in part for the comparatively rapid wear of the manganese steel, as was shown by the fact that, in the sand-blast test also, the manganese steel did not exhibit the superior wear-resisting qualities that it develops in service.

The reason for the comparatively rapid wear of the manganese steel in all of these tests is that severe cold-working must be associated with the abrasion, if manganese steel is to develop its maximum resistance to wear. The material in the heat-treated condition has a Brinell hardness of but 180 to 200, and when, as in these tests, it is subjected to pure abrasion, it wears but little more slowly than carbon steels of similar hardness. Under severe cold-working, however, the hardness of the surface of a manganese steel article, to a depth of about $\frac{3}{16}$ in., is rapidly raised to about 550 Brinell, giving it the power of resisting penetration by the particles of the material being crushed, so that it wears very slowly. In types of wear, such asthat to which stone crusher jaws are subjected, in which the abrasion is accompanied by severe pressure, the new surfaces successively exposed as the old surface slowly wears away are automatically hardened by the heavy pressure, so that the hardness of the surface is constantly maintained at about 550 Brinell. The rate of wear throughout the life of the casting, therefore, is very low.

To work out a testing machine in which the metal was both abraded and hammered or squeezed appeared at that time a well-nigh hopeless task, and we reluctantly concluded to adopt the slow and tedious method of crushing known amounts of hard stone between the jaws of a small laboratory stone crusher and determining the loss in weight of the jaws at regular intervals. A description of our experiences with this method, which exactly duplicates at least one kind of wear encountered in service, forms the major portion of this paper.

A year or two ago we again endeavored to find a more rapid method of testing manganese steel for wear, only to find that no machine has yet been developed that simulates any of the particular kinds of wear that manganese steel so successfully withstands in service. Tested in the Amsler machine, at the U. S. Bureau of Standards, manganese steel wore no better than carbon steel, because the test is one chiefly of abrasion. In tests in a small tumbling barrel or ball mill, also made at the Bureau of Standards, manganese steel specimens, even if previously hardened by cold-working, wore at about the same rate as test pieces of carbon steel, probably because abrasion far outweighed cold-working in the test. Indeed, our experience would lead us to expect such a result, for though manganese steel is unequaled as a material for ball-mill linings, especially when coarse rock or ore is being crushed, it has uniformly failed to pay for itself as a material for the balls themselves, compared to the hard carbon or chromium steels.

A machine developed by Brinell was next tried, in which the specimens were pressed against a rapidly revolving disk of soft steel or fiber, with a stream of sand directed against the point of wear to form the abrasive. From our previous experience we were not surprised to learn that in this test, also, manganese steel wore little better than carbon steels, even when the surface of the test pieces had been previously hardened by cold-working. Upon learning the results of this last test, we at first had visions of developing some machine that would simultaneously cold-work and abrade our specimens, but finally turned again to the tedious but reliable jaw crusher.

The two laboratory crushers used by us from 1907 to 1912 had fixed jaw plates of 20-sq.-in. surface area, and moving plates of 23.5-sq.-in. area. After no more than the usual amount of substitution of steel parts for cast-iron parts, the crushers were found capable of crushing, per 8-hour day, some 800 lb. of hard trap rock from $\frac{3}{4}$-in. size to dust and were run at this rate for over four years without undue expense for maintenance. A year or two ago we attempted to secure duplicates of these machines only to find that they were no longer manufactured. The slightly larger crusher now made by the same company proved inadequate to the task. Its design was different from that of our 1907 machines, and under steady operation its bearings would not stand up.

A machine of another make was then tried, only to find that the iron casting carrying the moving jaw plate broke after very short service. It was replaced by a steel casting, whereupon, the iron main frame of the machine gave way. A steel frame was then substituted with fairly successful results, the only drawback being the periodical failure by fatigue of the shaft upon which the swinging jaw oscillates. The designer of the machine placed two key-ways in this shaft to engage keys, whose function is to prevent end motion of the shaft, and regardless of the shape of the keys and key-ways used, the shafts fail at fairly regular intervals. This in a way is useful, as it enables us to check in service the conclusions derived from tests with a Farmer-type fatigue testing machine, but at times it is decidedly annoying, and were our tests to be run for an extended period, we would redesign the crusher so as to eliminate those two troublesome keys.

TABLE I.—DETAILS OF INDIVIDUAL JAW TESTS.

Analysis of Steel, per cent				Jaw	Area, sq. in.	Weight of Stone Crushed, lb.	Loss in Weight, g.	Loss in Weight per 1000 lb. of Stone Crushed, g. per sq. in. of Jaw Surface	Condition of Jaw
Carbon	Silicon	Manganese	Nickel						
1.21	0.32	11.9	....	Moving	20	4 000	13.5	0.16875	Good
						2 500	5.0	0.10000	Good
						4 000	8.0	0.10000	Peened (required grinding)
						4 000	6.5	0.08125	Good
						2 300	4.7	0.10217	Good
						4 000	7.3	0.09125	Good
						2 000	3.0	0.07500	Peened (required grinding)
						2 300	4.0	0.08700	Good (test discontinued)
						25 100	52.0	0.13600	
1.21	0.32	11.9	....	Fixed	23.5	4 000	16.0	0.1703	Good
						4 000	9.0	0.0958	Good
						700	3.0	0.1824	Good
						1 800	5.0	0.1184	Good
						4 000	11.0	0.1170	Good
						4 000	9.0	0.0958	Good
						3 800	8.0	0.0895	Good
						4 500	9.0	0.0851	Cracked
						26 800	70.0	0.1111	
0.54	0.52	0.65	....	Moving	20	4 000	94.0	1.175	Good
						3 800	55.0	0.723	Good
						4 100	56.0	0.682	Good
						4 000	57.0	0.712	Good
						4 000	50.0	0.625	Good (test discontinued)
						19 900	312.0	0.784	
0.54	0.52	0.65	....	Fixed	23.5	2 900	76.0	1.115	Good
						4 000	70.0	0.744	Good
						4 000	79.0	0.830	Good
						1 100	19.0	0.735	Worn out
						12 000	244.0	0.865	
0.55	0.21	0.90	.29	Moving	20	4 000	50.0	0.625	Good
						4 000	28.0	0.350	Good
						4 000	30.0	0.375	Good
						4 000	47.0	0.588	Cracked
						16 000	155.0	0.484	
0.55	0.21	0.90	3.29	Fixed	23.5	4 000	52.0	0.553	Good
						4 000	34.0	0.362	Good
						4 000	29.0	0.308	Cracked
						12 000	115.0	0.408	

TABLE II.—SUMMARY OF RATE OF WEAR OF VARIOUS STEELS.

Kind of Steel	Wear of Jaw Surface per 1000 lb. of Stone Crushed, g. per sq. in.	
	Fixed Jaws	Moving Jaws
12 per cent Manganese	0.0719 to 0.191	0.086 to 0.124
0.24 per cent Carbon	1.262	1.210
0.50 per cent Carbon	0.953	1.071
0.26 per cent Carbon, 1.23 per cent Chrome	1.19	0.821
0.59 per cent Carbon, 0.73 per cent Chrome	0.718	0.823
0.24 per cent Carbon, 3.63 per cent Nickel	0.577	0.467
0.55 per cent Carbon, 3.29 per cent Nickel	0.432	0.484
0.46 per cent Carbon, 6.24 per cent Nickel	0.236	0.270
0.44 per cent Carbon, 1.38 per cent Nickel, 0.44 per cent Chrome	0.406	0.391

In the tests run twenty years ago, it was our usual practice to weigh the jaws after each 4000 lb. of stone crushed, recording the loss of weight in grams, and the loss in grams per square inch of jaw surface per 1000 lb. of stone crushed. To ascertain the average loss figure at the end of the test, the total loss in grams per square inch of jaw surface was divided by the total number of thousand pounds of stone crushed; this figure is recorded for each jaw in Table I, and is the figure used in Table II. The figures in these tables were selected to illustrate certain conclusions arrived at as a result of our work.

It will be noticed that the moving jaw generally lost more weight than the fixed jaw; in the crusher we are now operating, which has somewhat larger jaws than those of our old machines, this condition is so pronounced that the fixed jaw hardly wears at all, and our conclusions are based chiefly upon the performance of the moving jaw.

When the jaws are first put to work the wear is far more rapid than it is later. This is due in all the specimens to surface decarburization of the steel and, in the case of the manganese steel at least, to the fact that the metal at first wears more rapidly than it does after its surface has become work-hardened by the pressure of the stone being crushed. For this reason, no test in which only a few hundred pounds of stone are crushed is of any value whatever, for it can easily happen that a steel which on extended test would give excellent results wears rapidly at first on account of a decarburized skin, while another, inferior steel, so treated as to be less decarburized at the surface, would at first appear to be better than the good steel whose surface was decarburized. Stone crusher tests have been brought to our attention in which the total amount of stone crushed was less than 2000 lb., and elaborate claims were made for one of the steels tested. In the light of the knowledge acquired in our own work we refused to accept the value of these data as a measure of the relative wearing properties of the steels tested.

Table I gives the details of the tests on two manganese-steel jaws, two plain carbon-steel jaws, and two nickel-steel jaws. Table II is a summary of our conclusions as to the relative rate of wear in stone crusher jaws of manganese steel, carbon steel, chromium steel, and nickel-chrome steel. In all cases the jaws are of cast steel. Manganese steel outwore carbon steel about 10 to 1; chromium decreased the wear of the pearlitic steels little if any; nickel decreased the wear to a marked extent. We do not claim a high degree of accuracy for our results, but as a guide to the behavior to be expected from various kinds of steel in resisting types of wear similar to that of stone crusher jaws, the data secured have proved of great value to us.

[For Discussion on Wear Testing of Metals, see page 348.—ED.]

THE WEARING QUALITIES OF TIRE CHAINS

BY W. H. PARKER[1]

SYNOPSIS

The paper describes a wear test developed primarily to study the wear-resisting properties of steel for tire chains. The specimen, which may be of almost any shape, including formed chain links, is held at constant pressure against a rotating alundum grinding wheel, the surface of which is kept continuously dressed. The number of revolutions required to produce a given depth of wear are automatically recorded.

The service to which tire chains are subjected involves the relation between impact and wear resistance, and a somewhat theoretical discussion of this relation as applied to the links of a tire cross chain is given. Some results of wear tests are reported. A comparison with service tests indicates that the laboratory test rates various materials in practically the same order as road tests, while giving a much greater distinction between them. The laboratory test furnishes a relatively quick means for comparing different materials and heat treatments.

INTRODUCTION

The difficulties of measuring the wearing qualities of metals in the laboratory have been discussed by several authorities and most of them agree that at the present time there is no general wear test that can be related to all kinds of wear. Each particular problem seems to require a separate solution, and that solution will be more nearly approached as the method of testing approaches the actual conditions under which the material concerned is used in service.

An attempt is made in this paper to outline one particular wear problem, to define the methods by which we have undertaken to solve it, and to present some test results and their relation to service.

The problem deals with the determination of the wearing qualities of automobile tire chains. Such chains consist of two parts,—side chains and cross chains. While both of these parts are subjected to severe service conditions, the cross chains present the most important and at the same time the most complex problem, and will be exclusively dealt with in this paper.

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[1] Research Engineer, American Chain Co., Bridgeport, Conn.

An analysis of the stresses to which a cross chain is subjected in service shows that the principal qualities which a cross chain needs for a long life are resistance to wear and resistance to impact. From a standpoint of service all other properties are negligible in comparison with these two, although certain other factors enter into commercial production, such as forming qualities, weldability, etc.

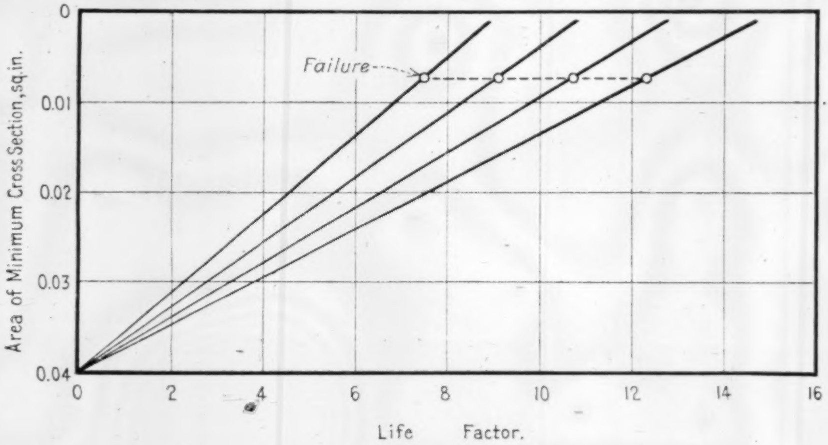
For the purpose of investigating the possibilities of different materials and studying the effect of heat treatments, some method of determining comparative resistances to these two stresses was needed. Having in mind the previous statement that the best results from a wear test could probably be obtained by a close approximation of service conditions, attempts were made to measure cross-chain wear on a road test. Since a cross chain always fails at the middle link, this receiving a maximum of both wear and impact, and since this link which fails can never be recovered, relative wear on a road test cannot be determined if the chains are run to failure. The only possible means of comparison is by running all chains an equal number of road miles and then endeavoring to measure their relative wear.

It has been found that this method is incapable of anything except the most broad differentiation. Measuring wear by the loss of weight of the middle link, the maximum difference shown between a very soft and a very hard material is about 8 per cent. The probable errors usually amount to more than the differences shown by the specimens.

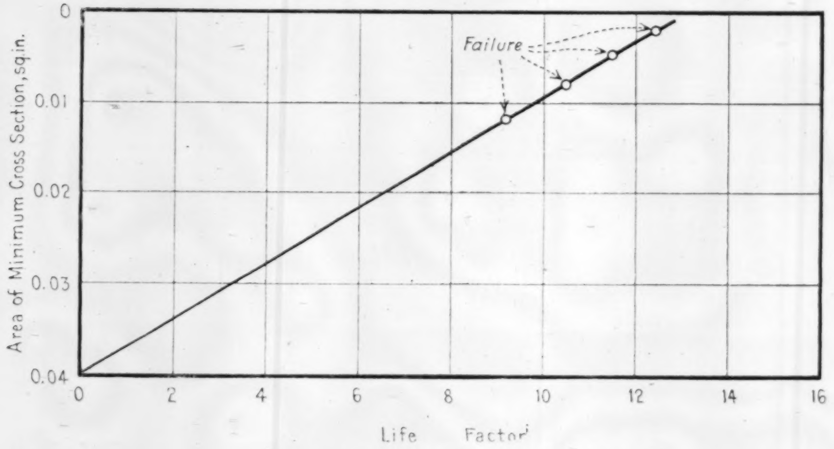
RELATION BETWEEN IMPACT AND WEAR RESISTANCE

The result of measurements of road wear and the small differences between materials which these show would seem to indicate that the resistance to wear of a material makes little difference in its service life, and that therefore impact resistance must be the main factor. The logical material for maximum service then would be that material with the greatest impact resistance. This conclusion is known to be in error from the fact that a case-hardened material, for example, will give considerably more service than the same material not case-hardened, although the case-hardening reduces the impact resistance to a large degree.

To ascertain the real relation between impact and wear resistance certain theoretical cases will be discussed. Consider the center links of a group of cross chains undergoing a road test. At any certain speed of the car, these links are subjected to a certain impact force, P, every time they strike the pavement. Neglecting the comparatively small change of weight due to wear, this impact force is constant for every link from the beginning of the test until the failure of the link. Each of these links has a certain definite unit impact resistance, S. As a link wears, the minimum section of the link decreases, and when



(a) Materials with equal impact resistance.



(b) Materials with equal wear resistance.

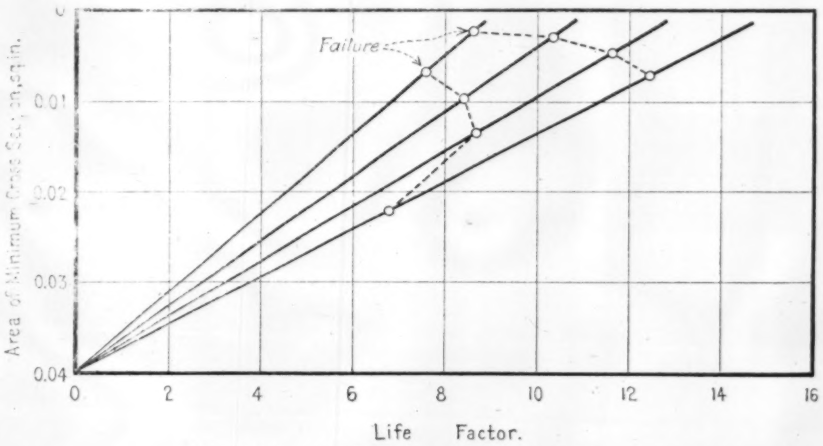
FIG. 1.—Theoretical Curves Showing Relation Between Wear and Impact Resistance.

this minimum section reaches a value determined by P/S, the link will fail in impact.

There are then two factors to consider: the length of time (or number of miles or number of revolutions, etc.), for the link to arrive at section P/S, and the value of S, since P is a constant. The first of these factors is a function of the resistance to wear and the second a function of the impact resistance.

In Fig. 1 (*a*) are shown four curves which represent wear tests of four materials, the area of the minimum section of the link being plotted against a life or mileage factor. Now consider these four materials to have an equal impact resistance. Then they will all fail at the same minimum section, P/S being equal, and their life will be in direct proportion to their wear resistance.

In Fig. 1 (*b*) are shown curves for four materials which are assumed to have an equal wear resistance. Having an equal wear resistance, they will each arrive at the same minimum section at the same time, but the value of P/S being different, they will fail in direct proportion to S, or the impact resistance.



(*c*) Materials with different wear and impact resistance.

FIG. 1.—Theoretical Curves Showing Relation Between Wear and Impact Resistance.

If now the two conditions are combined, there will be four materials, each with different wearing qualities and different impact resistance. The material which will give the longest life will be that material which gives the greatest amount of wear before arriving at a section where it will fail in impact. Speaking rather generally, it may be said that wear resistance and impact resistance are opposite properties. If a material with a certain impact resistance and certain wearing qualities is treated to give increased wear it usually becomes more brittle; so that in the case of the four materials the one which has the greatest wear resistance is weakest in impact. ¶ If the four curves of Fig. 1 (*c*) are the measure of wear and the unit impact resistance S of each material is known, the life of each material can be calculated as shown in the figure. Values for S may be determined by a standard impact test on round bars, and values of P are assumed.

It is seen that the material which gives the longest service may be neither the material which is the most resistant to impact nor most resistant to wear. It may also be noticed that the material which shows to best advantage is dependent upon the assumed value of P, which in turn is dependent upon the speed of the car. Therefore, a material that will give the best service at one speed may give the least service at another speed. Also, two materials that differ only slightly as to wear resistance may differ to a much larger degree as to life.

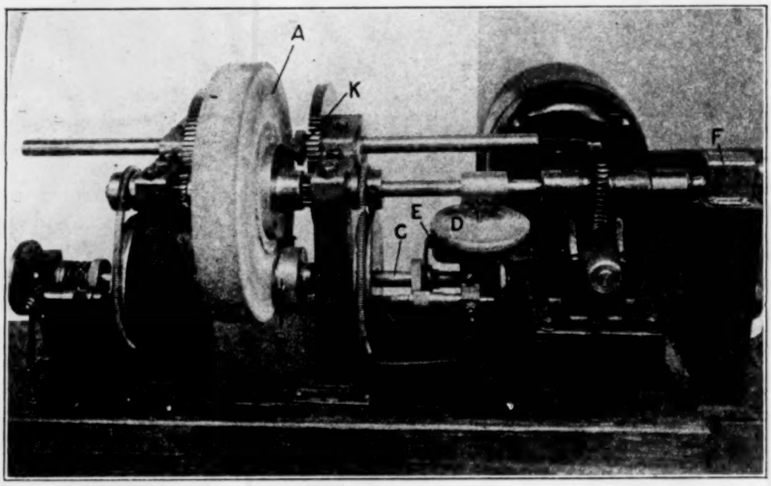


FIG. 2.—Abrasion Machine.

Now if by some means a progressive measurement of the wearing properties may be obtained, these measurements may be related to the impact test results as shown. These measurements must be such as to furnish a reliable comparison to actual service conditions, at the same time being sufficiently sensitive to differentiate between materials of various kinds.

WEAR TEST APPARATUS

An apparatus was developed for subjecting materials to wear in a manner similar to service. This apparatus is shown in Fig. 2. An 8-in. alundum grinding wheel A is rotated at 36 r.p.m. by a motor through a worm gear. The specimen is set with soft mounting metal into a holder B, which is attached to one end of shaft C. This shaft is driven by a spring belt from the main drive shaft at one-third the main shaft speed. The specimen is held against the grinding wheel with a constant load which is applied by a weight D acting through a lever E. Revolutions of the drive shaft are recorded by the counter F.

To the end of shaft C is fixed a brass disk G. This disk rotates with the shaft, and as the specimen wears, the disk moves toward the micrometer H, which may be set for any predetermined amount of wear. Contact between the disk and the micrometer operates a magnet to automatically stop the machine. At the rear of the machine is a grinding attachment K to clean the surface of the wheel. This consists of a tubular dresser rotated by an eccentric cam driven from the main drive shaft. The connecting gears are provided with an odd number of teeth so that the dresser does not trace the same path across the face of the wheel. The machine is built so that both sides of the grinding wheel may be utilized at the same time.

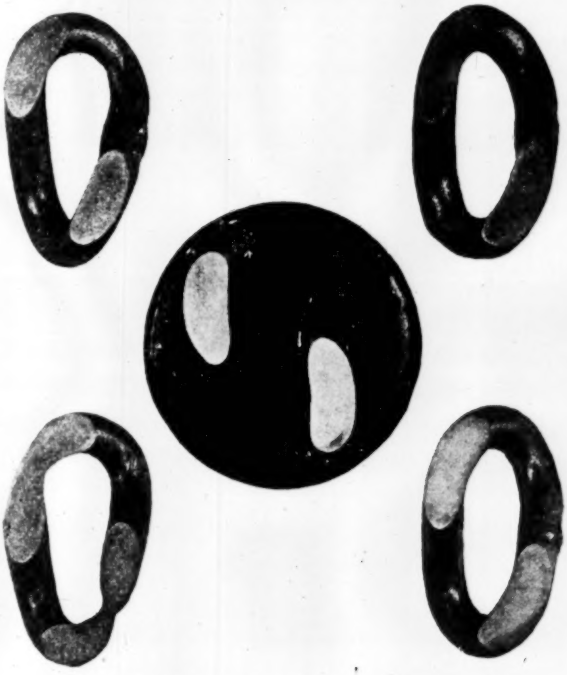


FIG. 3.—Specimens Tested on Abrasion Machine.

The specimens may be of almost any shape, the usual ones being round bars, tested either sideways or endways, or formed chain links. Figure 3 shows the manner of mounting specimens and also the appearance of some worn samples. A special jig is used to hold such samples while mounting to insure alignment.

The load which can be applied to the specimen on this particular apparatus is somewhat limited, being from ½ to 1½ lb. The limiting factors are sufficient load to overcome the inertia of the slip-pulley which drives the shaft *C* and allow the load to act on the specimen, and a load which does not cause the specimen to wear the grinding wheel faster than the dressing arrangement. Slight changes in the design would permit a wider range of loading. A load of 1 lb. is sufficient to test a chain link in about 12 hours.

A non-constant abrasive surface, a source of error in some forms of wear testing machines, has been practically eliminated in this apparatus by continuous dressing of the grinding wheel.

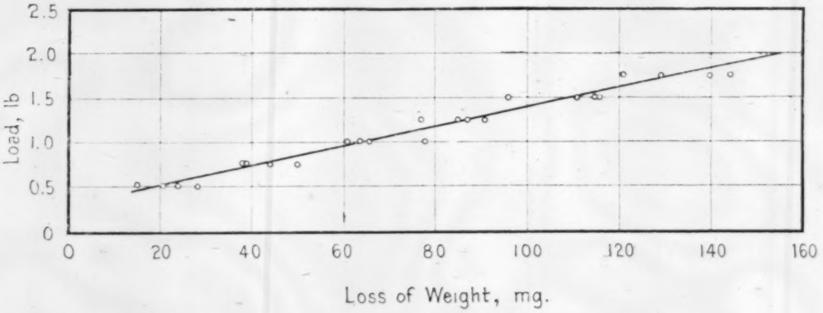


FIG. 4.—Relation Between Load and Wear on Abrasion Machine.

SOME RESULTS OF WEAR TESTS

Figure 4 represents a series of tests made to determine the effect of load, and also shows the reproducibility of results. The specimens used were of ¼-in. diameter drill rod. A number of specimens were run for the same number of revolutions with various loads and their loss of weight measured. These specimens were run without any intermediate preparation of the grinding wheel surface. These results, as well as numerous others, show that the abrasive surface does remain fairly constant. The continued dressing does wear out the grinding wheel in a rather short time, but by using a standard commercial grinding wheel a sufficiently close duplication can be made.

When chain links are tested curves similar to those shown in Fig. 5 may be obtained. In this case the automatic stop arrangement is used. Due to difficulty in keeping the face of the grinding wheel in accurate alignment this arrangement is used only for approximating a certain amount of wear, accurate measurement being made with the micrometer after turning both grinding wheel and specimen holder to the same relative position each time. The measurements thus made include both the wear of the specimen and the grinding wheel. The wear of the grinding wheel is dependent only on the pressure on the dresser and hence is fairly constant, so that it may be neglected in comparative tests, or it may be corrected for.

TABLE I.—COMPARISON OF MACHINE AND SERVICE WEAR TESTS.

MATERIAL	LOSS IN WEIGHT, PER CENT, SERVICE TEST	LOSS IN WEIGHT, PER CENT, MACHINE
No. 1	5.0	14.4
No. 2	7.7	19.6
No. 3	8.4	26.0
No. 4	8.6	23.7
No. 5	9.1	27.0

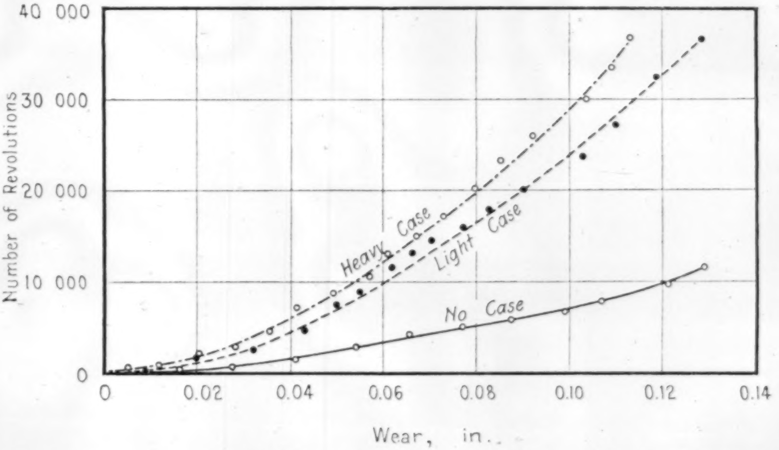


FIG. 5.—Representative Results of Chain Links Tested on Abrasion Machine.

Figure 5 illustrates somewhat the sensitivity of this apparatus. The relation between the amount of wear and the number of revolutions is shown for three chain links, all of the same material. One was untreated, one had a thin carburized case, and one a thick case.

A rough check on the ability of this apparatus to produce wear similar to service is obtained by comparing the loss in weight of cross-chain links worn on the road with the loss in weight of similar links worn on the machine, as shown in Table I. The percentage loss in weight is based on the total weight of the link. The shape of these particular links was such that a 33-per-cent loss in weight would cause failure by the wearing through of the link. On the service test each material was run for the same number of miles on the road and on the machine test each material was run the same number of revolutions. It may be seen that the machine rates these materials in practically the same order as the road tests, while giving a much greater distinction between them.

The rather ideal conditions under which the laboratory determinations of wear and impact resistance are made are not of course obtained in service or service tests. Hence the checking of the results of laboratory tests of any particular material with road tests is somewhat difficult. For some materials there may be certain other factors which have an influence upon the life of the cross chain, such as liability to failure under repeated impact, or tendency of certain sections of the link to develop cracks during heat treatment. These can be determined to some degree and allowed for in results.

In general it has been found that the correlated results of such laboratory tests give a reliable index of the service qualities of cross chains. They furnish a relatively quick means for comparing materials, heat treatments, etc., and have been particularly valuable in the preliminary selection of samples for long-time service tests.

WEAR TESTING OF VARIOUS TYPES OF STEELS

By J. M. Blake[1]

Synopsis

The paper describes apparatus developed to test the resistance of metals to abrasion under so-called "wet-grinding" conditions. In principle, the machine is a very accurately controlled grinding machine in which all conditions as to abrasive, pressure, speed, time and moisture or other liquid content are under the control of the operator. It is possible to approximate any given set of field requirements. The wear is measured by the amount of metal worn off in approximately 15,000 ft. of travel at a speed of 125 ft. per minute, using a pressure of 50 lb. per sq. in. on a testing face of approximately 3 sq. in., and is expressed as the ratio of wear loss as compared with a standard (commercially pure iron) at 100. By varying the abrasive used, it has been possible to develop alloys specifically adapted to particular field requirements.

The products of the company with which the author is connected are used to a very large extent in the handling or conditioning of abrasive non-metallic materials as in the various operations of mining, quarrying, road building, ditch digging, sand handling and the like, and the peculiarities of field requirements groups our products under three general classifications:

1. In the heavy-duty handling of non-metallic materials as with steam shovels, jaw crushers and the like, manganese steel has proved superior in most cases to any other type of steel, whether straight carbon or alloyed, and its use for work of this kind is almost universal. In applications of this nature, relative wear resistance is determined by full-scale field installations.

2. Where very high tensile or transverse stresses are encountered in field service, it is necessary to employ some of the especially heat-treated alloy steels. In this case, however, considerable wear resistance usually has to be sacrificed in order to secure great elastic strength, These heat-treated alloys include the nickel-chromium, low-manganese. chrome-molybdenum and other specially heat-treated alloy steels, and our method for determining relative wear resistance of such steels is by actual full-scale field installations.

3. In a third type of service neither heavy stresses nor heavy

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[1] Engineer of Tests, American Manganese Steel Co., Chicago Heights, Ill.

impacts are encountered, but high abrasion resistance is required under so-called "wet-grinding" conditions.

For reasons not as yet entirely clear, there is a great difference between the abrasion resistance of a steel under wet-grinding conditions on the one hand and dry-crushing conditions on the other, and materials which serve very satisfactorily for dry grinding or dry crushing may not be at all lasting under wet-grinding conditions. The element of oxidation or corrosion has been found to be a very important factor in this connection, but this cannot explain altogether the much more rapid wearing away of practically all metals under wet-grinding conditions as compared with dry grinding.

A very satisfactory line of steels exists for heavy duty under dry grinding or dry crushing conditions but these do not serve satisfactorily for wet grinding. A great need has existed for materials which are more abrasion-resisting under the latter conditions, that is, the handling of moist abrasives, and one of the most active phases of our studies has been directed toward the development of such materials.

In order to simplify the discussion of a subject which is very complicated at best, it is proposed to consider herein only abrasive wear of non-metals against metallic surfaces under wet or moist conditions and under relatively low pressures, and to describe specifically the means now utilized for measuring the relative abrasion resistance of various metallic substances under these conditions. It is not proposed to consider the voluminous literature which deals with the history of this subject, nor to discuss methods or machines which have been proposed for this purpose from time to time in the past. When our investigations were undertaken several years ago, all of the available methods and machines were very carefully considered and discarded as being not adaptable to these general requirements. The abrasion testing machine employed was developed by F. A. Fahrenwald and has been in continuous use for over two years, during which time determinations have been made on more than 1000 samples. The relative abrasion factors developed by this machine for common and known materials has shown such a close concordance with the relative performance of these same materials in field service that we feel justified in assuming that similar abrasion factors secured for unknown alloys can be safely interpreted in terms of their probable field performance.

Most of the special alloys which have shown very high abrasion resistance values on this machine have been tested under field conditions and it has been found that their listing in accordance with abrasion factors determined on the machine corresponds very closely to a listing in accordance with relative wear resistance under field conditions. It must be understood, of course, that physical properties other than mere wear resistance may determine the value of any given alloy for a certain industrial use; but a material which shows a low abrasion factor on this machine, coupled with necessary values for ductility and physical strength, will invariably show long life under abrasive field conditions.

In principle, this abrasion testing machine is a very accurately controlled grinding machine in which all conditions as to abrasive, pressure, speed, time and moisture or other liquid content are under the control of the operator. These conditions can be varied so as to approximate any given set of field requirements. It has been found that the relative position of any steel on the abrasion factor classification is not changed by variations in pressure or speed within limits which do not introduce complications due to increased temperature and the like.

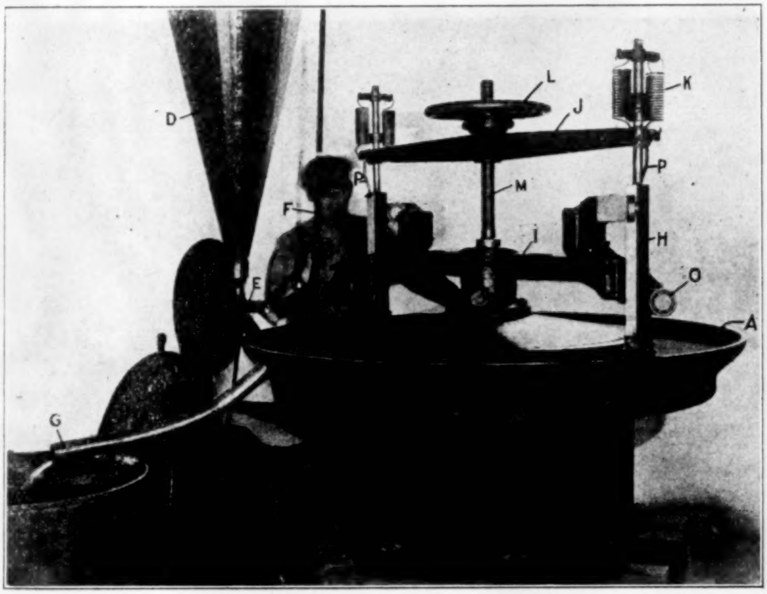

FIG. 1.—Apparatus Developed for Wear Testing of Metals Under Wet-Grinding Conditions.

Figure 1 shows a view of the machine during the course of a test. Figure 2 shows the details of construction. The essential units of the abrasion test are the annular trough *A* for containing the abrasive, the abrasive-holding surface, *B* (in this case a pure copper ring), and the specimen itself, *C* (Fig. 2). The abrasive is fed uniformly and continuously from a hopper *D* by means of the worm and spout mechanism *E*, which is driven in train with the testing machine itself. Water is metered through the pipe *F* to the trough. Pipe *G* carries the overflow of excess water together with fines from the disintegrated abrasive, and this insures against excessive accumulation of temperature and at the same time provides for continuous automatic conditioning of the abrasive material itself.

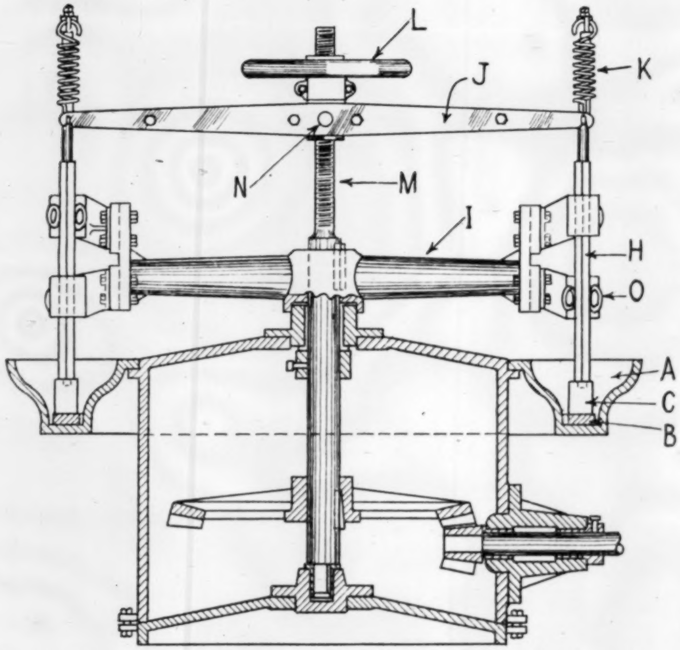

FIG. 2.—Showing Details of Construction of Wear Testing Apparatus.

The test specimens are attached to the vertical holders *H*, which in operation are forced to rotate in a position both perpendicular to the plane of the trough and parallel to each other and to the vertical axis of the machine. These holders are free to move vertically between adjustable ball bearing rollers at each end of drive arm *I*, and are weighted downwardly by means of the pivoted beam *J* through springs *K*, which are the only point of contact between *H* and *J*.

The desired total load is provided by means of the hand wheel *L* threaded to the central drive shaft *M* and this load is equally divided between the two specimens by the knife-edge *N* upon which pressure arm *J* is pivoted.

The specimens with their holding arms *H*, are driven along the race in a perpendicular position by the rollers *O*, which are attached to each end of the cross-arm *I*, which in turn is driven by a motor through bevel and worm gears, clearly shown in Figs. 1 and 2.

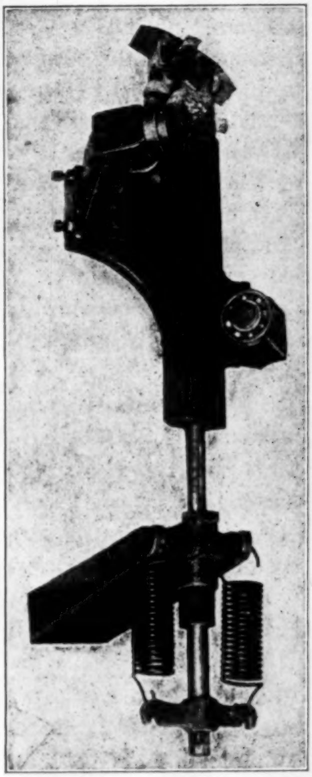

FIG. 3.—Showing Details of Specimen Holding and Operating Assembly.

Figure 3 shows in detail the specimen holding and operating assembly, with a typical specimen attached, but with the holder raised sufficiently to permit photographing.

The specimen engages the abrasive-coated copper surface at an angle of about 30 deg., this acute forward angle serving to funnel the powdered abrasive between the gliding surfaces. The abrasive is deeply indented into the soft copper ring which serves as the bond for holding and forcing the particles of abrasive across the surface of the specimen.

Pressure between specimen and abrading surface can be adjusted from zero to approximately 500 lb. per sq. in., and this pressure can be set or read without stopping the machine by means of the pointers *P*, which indicate direct readings on a scale (attached to *H*) which has been previously standardized against the springs in use.

The specimen chosen for our conditions is $\frac{3}{4}$ by $2\frac{1}{4}$ in. in section and may be of any convenient length over 4 in. The working surface is cast or cut to the approximate 30 deg. angle and each test involves a preliminary "grinding-in" so as to fit the specimen to the exact contour of the abrasive-holding copper surface.

This method of abrasion testing originally contemplated the continued use of a standard specimen in every test in order to check each unknown individually against the unit of comparison, but it was found that results for the standard remained constant regardless of what the companion test specimen might be, so long as the abrasive did not change; so this standard is now run only periodically to serve as a periodic check, which permits results on two unknown steels to be secured from each run.

The standard test is 1200 revolutions on the 4-ft. diameter ring or approximately 15,000 ft. of travel, at a speed of 125 ft. per minute. A pressure of 50 lb. per sq. in. on a testing face of approximately 3 sq. in. is customarily used.

The amount of metal worn off during a test may vary from 3 g. for some new very hard steels to 150 g. or more for alloys with a high Fahrenwald abrasion factor. The Fahrenwald abrasion factor for each steel is its ratio of wear loss as compared with the standard (commercially pure iron) at 100.

The abrasive constituent of most of the non-metallic materials encountered by our products is quartz in some form or other, as sand or gravel, or as the gritty portion of clay or loam. Other minerals are, of course, met with also, but these are all softer and supposedly less abrasive than quartz. In most of our tests, therefore, the relative abrasion resistance of new steels and alloys is determined with a standard controlled grade of silica sand as the abrasive. From time to time a special series of tests is run using abrasive material typical of some other type of field requirements, by simply substituting material secured from this field for the regular sand in the trough of the machine.

In a very consistent manner, this abrasion tester has shown that each abrasive or mineral substance develops its own order of relative abrasion resistance for metals and that the position of any steel or alloy on a listing of abrasion factors will depend upon the nature of abrasive employed. In extreme cases it has been found that steels at the top of the list with one abrasive are at the bottom of the list with another. This discovery has eliminated the idea of securing general all-around high abrasion resistance in any one steel, and has enabled us to develop alloys specifically adapted to particular field requirements.

The Fahrenwald abrasion factors against sand for several well-known materials, together with several new very hard alloys, are as follows:

Standard Specimen (commercially pure iron)	100
Merchant Bar Iron, Annealed	92
White Iron	88
Manganese Steel	77
Tool Steel, Hardened	76
White Iron, Chilled	72
High-Speed Metal, Hardened	57
Stellite (welded wearing face)	51
Amsco Alloy No. 459 (welded wearing face)	31
Amsco Alloy No. 459 (as cast)	10

The alloy listed as Amsco Alloy No. 459 is an alloy containing approximately 3.5 per cent of carbon and 15 per cent of tungsten group metals to which varying amounts of nickel and other toughening constituents may be added. The composition is varied quite widely depending upon industrial requirements.

Earlier attempts to rate the relative abrasion resistance of metals on the basis of loss in weight per unit of surface and time, proved very confusing because of the numerous variables involved and especially the tremendous variation in relative wear with different abrasives.

The Fahrenwald abrasion factor is substantially not a function of time, nor of pressure or speed as is any other method of rating, and provides even for the variation in *relative* wear with different abrasives by always listing the unknown in terms of comparative performance of a material well known to man in all its characteristics for ages.

DISCUSSION ON WEAR TESTING OF METALS

Mr. Gillett.

MR. H. W. GILLETT.[1]—I think it is quite important to notice in all these papers and in other work being done throughout the country that everybody realizes that wear testing is a difficult subject. The conditions of service vary. One cannot have any one wear test that will answer everything, but with the right type of test, one does get the right answer. You will note in all the papers, that the answer was known, pretty closely, and that the real task is to find a test that will give that answer.

In Mr. Blake's summary, it is quite interesting to notice that "Amsco" Alloy No. 459 (as cast), gives only 10 per cent of the loss on abrasion that pure iron does. But the statement is rather tantalizing from the point of view of the general study of wear-resistance because no information is given as to what its other properties are and what particular type of service it is used for. This alloy is evidently for pure abrasion, not the type Mr. Hall was dealing with, but I cannot quite believe that this is solely for wet grinding purposes.

Mr. Blake points out that the question of oxidation or corrosion under wet grinding conditions is quite important; it is obvious all through that one does not have any one single wear test; one has to consider the service conditions as well as one's material. People often write in to the Bureau of Standards, send in a piece of metal and want to know how long it would last in a given service under corrosion, particularly atmospheric corrosion. In such cases a perfectly good reply (though one not ordinarily phrased in this way) is that the metal is important and the atmosphere is important, and if they would send us a specimen of what the atmosphere is that is to come in contact with it for the next 20 years, we might be able to make a better guess at it. This case is analogous. After long years of endeavor, we know there is no single corrosion test, one needs to fit his corrosion test to the conditions of the service, one must use a little common sense in choosing the test conditions. Intensify the conditions slightly, if you will, but do not change the conditions of service. So far as I can see, wear testing for a long time to come is going to be long drawn out. We cannot safely accelerate it until we know what the answer is by long-drawn-out methods. If we can get

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[1] Chief, Division of Metallurgy, U. S. Bureau of Standards, Washington, D. C.

Mr. Gillett. a short cut which will give us the right answer on these materials for these conditions, then it is safe to assume that if we take new materials that we want to develop for that same service and give them that test, the cheapest way to get the information is to get it in the laboratory on a suitable wear testing machine for the purpose in hand.

Mr. Rosenberg. MR. S. J. ROSENBERG[1] (*presented in written form*).—In its research on the wearing properties of metals, the Bureau of Standards has experimented with several types of machines and in all cases the results of tests with any one machine agreed generally with the results secured by other investigators using the same type of test.

Recently the Bureau has studied the application of the Brinell machine to the wear of metals, particularly shovel blade materials. Generally speaking, this type of test showed very little difference between the various steels. Greater differences were found between check heats of approximately the same composition than between steels of different compositions or steels of the same composition treated differently. The conditions existing during the test apparently far outweigh in importance the wearing characteristics of the metal under test. Thus, a new wheel, even though previously abraded by running against a blank, shows any test steel to have a smaller wear resistance than that steel would show when tested after the wheel has been in service for some time. The sand used in the test also has a very marked influence on the wear. Specimens from the same lot of steel showed only about two-thirds the wear it had previously shown, when tested using sand which, according to the manufacturer, came from the same batch as the sand used in the first tests. An analysis of the sand used showed the grain size of the sand producing the least wear to be slightly smaller than the other.

As indicated in the paper by Mr. J. H. Hall, high-manganese steel, when tested in this machine, showed no superiority in wear resistance over the ordinary carbon steels. This is in accordance with common experience. Mr. Hall, however, used work-hardened manganese steel and stated that the hardness of work-hardened manganese steel is raised to about 550 Brinell to a depth of about $\frac{3}{16}$ in. This is considerably more than the greatest depth of wear ordinarily encountered in tests on steels in the Brinell machine (about 0.5 mm. or 0.039 in.). What reason can Mr. Hall suggest for his manganese steel not showing a relatively high wear resistance?

The Bureau in its tests of manganese steel in a ball mill has found it inferior to many steels in this test. Is this because the depth of abrasion exceeds the depth of the work-hardened skin? If not, why

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[1] Assistant Metallurgist, U. S. Bureau of Standards, Washington, D. C.

Mr. Rosenberg. does work-hardened manganese steel not show its superiority in this test?

Mr. Hall has stated that manganese steel, tested in the Amsler machine, wears no better than carbon steel because the test is one chiefly of abrasion. The reason is far from clear. As calculated by Hertz' formulas, the unit pressure existing between the manganese steel and the carbon steel against which it was tested varied between 58,000 and 74,000 lb. per sq. in., this stress being induced by a total pressure of 60 kg. The manganese steel tested had been heat treated to give high resistance to wear, namely, water quenching from 1800° F. The pressures under which the tests were made should have been sufficient to cause the manganese steel to work harden and thus exhibit its characteristic high resistance to wear.

From this it may be concluded that in at least three types of wear tests manganese steel does not show any degree of superiority over ordinary steels and that so far no satisfactory explanation of this has been offered.

Mr. Karelitz. MR. G. B. KARELITZ[1] (*presented in written form by Mr. J. M. Lessells*).—Referring to the paper by Mr. H. J. French, the large amount of work done by the author in his attempt to represent the wearing qualities of bronze bearing metals in a logical manner is very interesting, although as he states the tests do not characterize completely the properties which are of interest in the application of bronzes to bearings. The question of standard laboratory tests which would be really representative for bearing qualities of alloys is exceedingly important. To date such do not exist; the tests being used in various laboratories are only substitutes which are admissible only in the absence of better. The unavoidable boundary lubrication, at which the qualities of the bearing metal come into play, makes the conditions of testing very uncertain. The difficulty is met not only with bronzes; it has been encountered also in the work on babbitts which is now being conducted in the Westinghouse Electric and Manufacturing Co., where the work is directed in part toward finding an adequate method of laboratory testing for babbitts. A concerted effort in working out such methods would be of the utmost importance to the industry.

It is interesting to consider the difference in abrasion of the bronzes when tested dry and when lubricated in the light of the investigations published by Thomlinson in the *Proceedings* of the Royal Society, July, 1927. He studied the phenomenon of rusting of well-fitted steel surfaces. He proved quite convincingly that this

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[1] Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

Mr. Karelitz. rusting is caused by, and only in the presence of, sliding of the surfaces. He shows that molecules of steel are dislocated by the side pull of the attraction forces, in a manner similar to the technic of a dentist, these freshly dislocated molecules being readily attacked by oxygen. He has also shown that with greasy surfaces rusting can occur only under exceedingly high pressures. It appears that the picture of abrasion as a sidewise dislocation of particles fits in very nicely with the data of this paper: (1) Without lubrication the rate of arbasion is constant; (2) With lubrication the number of dislocated molecules is small; (3) Chill castings are worn much quicker than sand castings, since the particles are smaller and are easier to dislocate. Does this mean that a coarse-grained structure is always beneficial against wear?

With regard to proper interpretation of the test results, in view of the fact that lubrication is always present in bearings, even if inadequate at times, it is worth while to pay attention to the very remarkable results published by Mr. Suzuki of the Japanese Government Railways. He states that with various steels and various treatments he has found for a given material the abrasion is a very definite function of the coefficient of friction between the surfaces, irrespective if dry or lubricated. The abrasion varies as the square of the friction. Should this be true for bronzes as well as for steels, further light will be thrown on these test results.

Mr. Lessells. MR. J. M. LESSELLS.[1]—My remarks refer to the paper by Mr. French. It is unfortunate that Mr. French does not include any compression tests on a bearing metal subject to compression.

The second point I should like to bring out is that Mr. French has run these tests under dry conditions and those of complete lubrication. It probably would be interesting to obtain wear data under conditions of boundary lubrication. This is important because very few bearing metals will ever be subjected to dry conditions.

The third point I should like Mr. French to comment on is the duplication of his pounding test data. We have a pounding machine which we have used for some time on babbitt materials, but our chief difficulty has been the duplication of test results.

The paper covers a tremendous field and it is to be hoped that Mr. French will dig further into the subject and give us more information on it.

Mr. Moore. THE PRESIDENT (*Mr. H. F. Moore*[2]).—The Chair will ask the

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

Mr. Moore. privilege of making one statement about the Hertz formula. It is very true that in the Hertz formula the pressure between a cylinder and the plane of another cylinder is very great at the point of contact, and the stress measured as a pressure is very great in the vertical direction (assuming the axes of the cylinders to be horizontal), but there are very great compressive stresses in the two *horizontal* directions, the result being that the strains at the surface are not anywhere nearly so great as the stress would indicate, and that the shears on inclined planes are very materially reduced, so that it is quite possible that though this surface pressure is very great, the slip resulting in work hardening may be smaller than would be indicated by the pressure and the slip may actually be greater at some point below the surface than it is at the surface.

Mr. Herschel. MR. W. H. HERSCHEL.[1]—In reply to Mr. Lessells, I think the conditions of test in Mr. French's experiments are such that one will unavoidably have boundary lubrication, because one has both very high pressures and slow speeds of rubbing, both of which are the conditions under which one has boundary lubrication. If Mr. Lessells has any suggestions as to how these conditions could be changed, so that it would make it more certain that boundary lubrication would be obtained, I should be very interested to hear the suggestions.

Mr. Lessells. MR. LESSELLS.—In answer to this I have very little to offer. It may be possible to obtain boundary lubrication by suitable control of oil supply.

Mr. Herschel. MR. HERSCHEL.—I might point out that it is not a question of the amount of lubricant applied, but the amount of lubricant which actually penetrates between the two metals.

Mr. French. MR. H. J. FRENCH.[2]—In reply to Mr. Lessells, I should like to say that the reason that tension tests were used instead of compression tests is that the manufacturers are continually faced with specifications based on tension tests. We realize that the ordinary bearing is not under tension but is under compression and that compression tests would give useful data. However, with Brinell hardness tests, repeated pounding tests and tension tests, compression tests were not considered to be essential to a reasonable interpretation of the results of the investigation.

I cannot cite definite figures at the present time with respect to the duplication of results in the pounding tests, but I can say that from the work we have done, the order of duplication is partly dependent upon the length-diameter ratio of the test specimen; it is also

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[1] Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

Mr. French. quite largely dependent upon the methods used in preparing the test castings. We had less difficulty in the work with railroad bearing bronzes with relatively heavy sections than we have had in more recent work with bushings of relatively thin sections. Likewise, a length-diameter ratio of 2 in the repeated pounding test specimens has yielded somewhat more consistent results than a ratio of 2.5. Differences in the crystal structure from the outside to the center in thin bushings or plates resulted in non-uniform bulging of the pounding test specimens, but with suitable control of the diameter length ratio, we have obtained close concordance in results. Certainly there has been no difficulty in detecting with certainty appreciable differences in resistance to pounding or the general trends as the proportions of one element in the bronzes are varied throughout reasonably wide limits.

I should like to raise one question with respect to Mr. Karelitz' discussion in which he commented on the smaller particles in chill-cast metal than in sand-cast metal. I assume that by particles he means crystals. I should like to confirm what Mr. Herschel has said with respect to the boundary conditions of lubrication mentioned by both Mr. Lessells and Mr. Karelitz. We suspect but are not certain that our high-pressure tests approached conditions of boundary lubrication.

If Mr. Lessells or some one of his associates can suggest some method by which reproducible wear tests can be made under conditions which are definitely known to represent so-called boundary lubrication we shall appreciate a detailed description.

Mr. Hall. MR. JOHN HOWE HALL.[1]—In reply to Mr. Rosenberg, I think that in the paper I have perhaps described the surface hardening of manganese steel inaccurately, that is, I have chosen my words inaccurately. At the very surface the hardness is exceedingly high, but according to our measurements any effective hardness dies out in about $\frac{3}{16}$ in. From the wording in the paper it might be assumed that the hardness of the metal was 550 to a depth of $\frac{3}{16}$ in., and then abruptly changed to 180, which of course is not true.

The tests on the Brinell machine were made in Sweden. In the case of the specimens tested in the Brinell machine, I work-hardened some of them myself in the laboratory with a hammer, and those specimens and the un-work-hardened specimens were then sent to Sweden and tested there. We never saw the specimens after testing; I do not know how deep the notches made in the testing were. I

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[1] Metallurgical Engineer, Taylor-Wharton Iron and Steel Co., High Bridge, N. J.

Mr. Hall. assumed, as Mr. Rosenberg said, that I had hardened that material with a hammer deeper than the Brinell machine wears it. I think it probable that the reason why in the Brinell machine and in the ball mill, the effect of the laboratory work-hardening was not the same as the automatic work-hardening of service, is that we were too enthusiastic with the hammer. That is, we pounded the material so hard that the very surface was made quite brittle, and in the Brinell machine and possibly also in the ball machine, that over-hardened surface chipped off so fast that it did not give good results; it did not give the same results that are found on castings which are automatically hardened in the field.

I am aware that the theory that manganese steel owes its resistance to wear to the power of work-hardening is not a complete theory, but our experience has been that if the steel is found to be work-hardened by the service to which it is exposed, it will have a long life; and if it is not work-hardened by the service to which it is exposed, it will not have a long life. There are almost no exceptions to that rule, so that as Mr. Gillett said, in that particular case we really know part of the answer before we start our wear testing work; and if the theory will not square up with the practice, it is the theory that is wrong and not the practice. Now why, in the Amsler machine, under heavy pressure, the steel does not work-harden or give the results it might be expected to give, I am not prepared to say. I should like to ask Mr. Moore if his remarks about pressure applied to that.

Mr. Moore. THE PRESIDENT.—I should not know until I could see the details of the Amsler machine. I was referring more to the discussion which dealt with the great pressures set up at the area of contact of two cylinders.

Mr. Pike. MR. ROBERT D. PIKE[1] *(by letter)*.—The paper by Mr. H. J. French presents a considerable advance in our knowledge of methods of testing bearing metals.

Recently some very remarkable wearing and low anti-friction properties, demonstrated in practice, of an alloy of 75 per cent of copper, 25 per cent of lead, and containing no tin, have come to my attention. The alloy nearest to this composition tested by the author is 70 per cent copper, 24 per cent lead, 6 per cent tin, and his alloy containing the minimum of tin, namely, about 1 per cent, has the approximate composition copper 85 per cent, lead 14 per cent, tin 1 per cent.

It is to be regretted that the area of the copper-lead-tin ternary

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[1] Consulting Chemical Engineer, Emeryville, Calif.

Mr. Pike.

diagram shading off into copper 70 to 75 per cent, lead 25 to 30 per cent, and tin in minor amounts or absent has not been investigated by the author. In view of the fact that this area has not been investigated, the author's conclusion "bronzes with less than about 5 per cent of tin did not seem to have the combination of mechanical and wearing properties necessary for good service as bearings" is more general than justified by the supporting data.

The author brings out very clearly that all mechanical properties of all bronzes are impaired by raising the temperature. This being so, it would seem that one of the most important functions of a bearing metal is its power to dissipate heat by conduction. A study of the thermal conductivity of bearing metals would, therefore, probably shed considerable light on their adaptability to severe conditions of service.

THE INCREMENTAL PERMEABILITY METHOD FOR THE MAGNETIC ANALYSIS OF HIGH-SPEED STEEL

By W. B. Kouwenhoven[1] and Julian D. Tebo[2]

Synopsis

This paper describes a new method of magnetic analysis which the authors have called the incremental permeability method. It uses two magnetomotive forces simultaneously, and the change in induction produced by the superimposed or incremental magnetomotive force in the specimen is measured. This superimposed magnetomotive force may be produced by direct or alternating current.

The magnetic properties of high-speed tungsten steel bars were investigated using this method and data were obtained which makes it possible to differentiate between the heat treatments received by the specimens.

Introduction

The purpose of this investigation was to test a new method of magnetic analysis, which the authors have called the incremental permeability method, and to determine what relation, if any, exists between the magnetic properties as found by this method and the heat treatment of high-speed steel. The object was to correlate certain magnetic characteristics of the specimens with their heat treatments. The work was done in the electrical laboratory of the Johns Hopkins University, under the auspices of the Society's Committee A-8 on Magnetic Analysis. Previous investigations have been made by this committee on high-speed steel twist drills (**1**).[3] Numerous tests have been made by other investigators (**2, 3, 4, 5, 6, 7, 8**) on the specimens used in the present investigations.

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[1] Associate Professor of Electrical Engineering, The Johns Hopkins University, Baltimore, Md.

[2] Graduate Student in Electrical Engineering, The Johns Hopkins University, Baltimore, Md.

[3] The boldface numbers in parentheses refer to the papers given in the list of references appended hereto.

INCREMENTAL PERMEABILITY METHOD

The incremental permeability method of magnetic analysis uses two magnetizing forces simultaneously on the same specimen. A fixed magnetizing force of some predetermined value, F_F, is first applied to the specimen and then a second magnetizing force, ΔF, called the incremental force is superimposed upon F_F and the increment or change in magnetic induction, ΔB, produced by ΔF is noted. The fixed magnetomotive force, F_F, is produced by a continuous current. The incremental magnetomotive force, ΔF, may be positive, that is, in the same direction as F_F; or negative, that is, opposing F_F; or it may be an alternating magnetomotive force.

It is a well-known fact that the shape of the hysteresis loops for specimens of the same chemical composition depend to some extent upon the heat treatment received by the specimen. It is also true, however, that slight changes in the drawing temperature for specimens of a given quench often produce such small changes in the maximum induction, residual magnetism, and coercive force as to make it impossible to differentiate between two specimens which had received different heat treatments. It was therefore decided to attempt to separate specimens that had received different heat treatments by using superimposed magnetizing forces, or an incremental permeability method, as it has been named.

This investigation may be divided into the following sections:

1. Development of apparatus and procedure for applying the incremental permeability test to specimens.
2. Determination of the best values of constant magnetomotive force, F_F, and the superimposed magnetomotive force, ΔF, which will separate specimens whose hysteresis loops overlap at the cardinal points, by giving different changes of induction ΔB.
3. Analysis of the results.

SPECIMENS

The specimens used in this investigation were made from high-speed steel obtained from the Horne Steel Products Co., and the heat treating and preliminary work was done under the direction of Mr. Thomas Spooner (2), of the Westinghouse Electric and Manufacturing Co. The bars from which the specimens were cut were $\frac{1}{2}$ in. square and were supposed to be all from the same heat. Each bar was assigned a letter and then cut into specimens approximately 10 in. long, each specimen being given a number in the order in which it was cut from the bar. The markings on some of the bars could not be deciphered or had been omitted, and these bars were marked with the letter *X*. The specimens were ground to 0.45 in. (1.14 cm.) square and then normalized at 1650° F. (899° C.)

In the early magnetic tests of the normalized bars certain specimens marked *K* differed from the others. The cause of this was found to be due to a difference in the chemical composition of the *K* specimens as shown below:

	CARBON, PER CENT	SILICON, PER CENT	SULFUR, PER CENT	PHOSPHORUS, PER CENT	MANGANESE, PER CENT	NICKEL, PER CENT	CHROMIUM, PER CENT	VANADIUM, PER CENT	TUNGSTEN, PER CENT
Normal Material.	0.79	0.42	0.042	0.031	0.176	0.10	3.46	0.88	17.35
K Sample..	0.70	0.30	0.030	0.021	0.173	0.11	3.97	1.49	14.03

TABLE I.—HEAT TREATMENT OF BARS.

Drawn at		Quenched at 2109° F. (1154° C.)			Quenched at 2211° F. (1211° C.)			Quenched at 2316° F. (1268° C.)			Quenched at 2366° F. (1296° C.)			Quenched at 2419° F. (1326° C.)		
deg. Fahr.	deg. Cent.	Bar	Length		Bar	Length		Bar	Length		Bar	Length		Bar	Length	
			in.	cm.		in.	cm.		in.	cm.		in.	cm.		in.	cm.
No draw		B-2	10.00	25.4	A-10	10.06	25.7	K-3	9.53	24.2	D-6	9.00	22.9	J-4	10.00	25.4
		F-6	10.09	25.8	F-4	10.00	25.4	F-3	10.03	25.7	F-2	9.50	24.2	X-9	10.06	25.7
					K-4	9.47	24.1	A-12	10.06	25.7	A-4	9.82	24.9			
1000	538	C-6	9.41	23.9	L-7	9.50	24.2	L-6	9.44	24.0	B-5	10.00	25.4	I-5	9.50	24.2
		L-11	10.09	25.8	G-14	9.38	23.8	G-9	10.06	25.7	G-6	9.70	24.6	X-11	9.31	23.6
		G-5	9.44	24.0	B-4	10.06	25.7	B-10	9.41	23.9	L-1	9.60	24.4	K-2	9.47	24.1
1050	566	K-8	8.94	22.7	C-7	9.00	22.9	C-9	9.47	23.9	H-4	9.41	23.9	X-15	10.09	25.8
		X-5	9.50	24.2	H-10	9.47	24.1	H-8	10.03	25.7	X-6	9.38	23.8	L-4	9.44	24.0
		H-11	9.91	25.2	X-1	10.06	25.7	X-14	10.09	25.8	E-7	8.91	22.6	G-2	9.47	24.1
1100	593	D-14	10.00	25.4	I-10	9.97	25.3	D-7	8.40	21.3	X-12	9.44	24.0	H-5	8.91	22.6
		I-11	9.47	24.1	D-8	10.06	25.7	I-6	9.47	24.1	K-1	8.75	22.2	X-17	10.06	25.7
		X-23	10.06	25.7				X-16	10.06	25.7	I-1	9.87	25.1	X-2	9.53	24.2
1150	621	J-14	9.63	24.5	J-3	10.00	25.4	J-6	9.44	24.0	X-13	10.09	25.8	X-7	8.91	22.6
		E-10	10.03	25.7	X-26	10.03	25.7	E-6	10.06	25.7	C-5	8.89	22.6	X-19	10.00	25.4
		X-24	10.06	25.7	E-8	8.89	22.6	X-22	10.09	25.8				E-1	9.30	23.6

HEAT TREATMENT

After normalizing, 75 of the bars were quenched, 15 for each of the following temperatures: 2109° F. (1154° C.), 2211° F. (1211° C.), 2316° F. (1269° C.), 2366° F. (1296° C.) and 2419° F. (1326° C.). Twelve bars of each quench were then drawn in lots of three each at the following drawing temperatures: 1000° F. (538° C.), 1050° F. (566° C.), 1100° F. (593° C.) and 1150° F. (621° C.). In the final state there were thus three bars for each pair of drawing and quenching temperatures.

The heat treatments were made in a salt bath under carefully controlled conditions. The mean temperatures are believed to be correct to within ±3° F. (1.5° C.) with a maximum variation of ±8° F. (5° C.), the average variation being less than half of this amount.

The heat treatment was completed approximately eight months before the work described in this paper was started and it is safe to assume therefore that there should be no uncertainty due to aging effects.

The bars available for this investigation and their heat treatments are given in Table I.

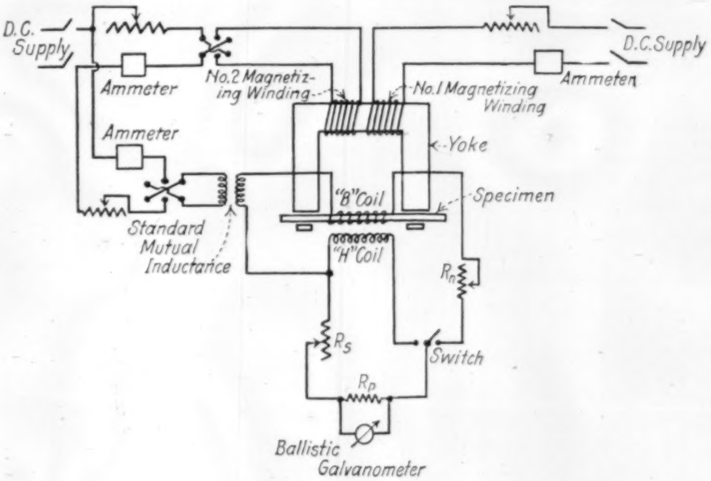
D.C. Supply
D.C.Supply
Ammeter
No.2 Magnetizing Winding
No.1 Magnetizing Winding
Ammeter
Ammeter
Yoke
"B" Coil
Specimen
Standard Mutual Inductance
"H" Coil
R_n
R_s
Switch
R_p
Ballistic Galvanometer

FIG. 1.—Diagram of Connections for Direct Current Tests.

APPARATUS

Magnetic and Electrical Tests:

The magnetic tests applied to the specimens were as follows:

1. Continuous current positive incremental test.
2. Continuous current negative incremental test.
3. Alternating current incremental test.
4. Differential alternating current incremental test.

In addition, the resistance of some of the bars was measured using a Kelvin Double Bridge.

A special permeameter of the Fahy Simplex Type was constructed for use in the positive and negative continuous current tests. This special permeameter had two separate magnetizing windings as nearly alike as possible wound on a laminated silicon steel core. The connections are shown in Fig. 1, where F_F is furnished by a 120-volt battery and ΔF by a 6-volt battery. It is important that the constant F_F be furnished by a relatively high voltage source with sufficient resistance in the circuit to limit the current to the desired amount. Under these conditions the application of ΔF makes only a small change in F_F and reduces the secondary inductive effects to a negligible amount. Oscillograms of the current and voltages induced in the circuits by the application of ΔF confirmed this statement.

In the alternating current incremental test a special solenoid 3 in. long was wound. This solenoid had two magnetizing windings each of 500 turns of No. 18 B. & S., D. C. C. wire and a third winding of 6000 turns of No. 36 enameled S. S. C. wire which served as a search coil. The core opening was ½ by ½ in. and fitted the specimens closely.

A special differential magnetic tester similar to that devised by Fahy(9) was constructed for the differential incremental tests. This tester had, however, two magnetizing windings which were similar to those provided for the simplex tester described above. Tests of this permeameter showed that the two halves were nearly equal. Values of induction measured on the same specimen in the two halves of the instrument checked within 50 gausses in 10,000.

POSITIVE INCREMENTAL METHOD

The normal magnetization curves and the hysteresis loops of a number of different specimens were taken and examined for the most favorable point at which to apply the incremental magnetomotive force. These showed that the steepest part of the normal induction curve for the specimens occurred at a magnetomotive force of about 40 gilberts per cm. Tests were made, however, at values of F_F ranging from 30 to 75 gilberts per cm. and with a ΔF from 4 to 30 gilberts per cm.

A specimen was first demagnetized and then the F_F was applied through winding No. 2 of Fig. 1, reversed twelve times to bring the specimen into a cyclic condition and the induction, B, measured. Then the incremental magnetizing force, ΔF, was applied through No. 1 winding of the permeameter and the change in induction, ΔB, noted.

In the positive incremental method the maximum separation was found at a constant F_F of 38 gilberts per cm. and with a ΔF of 6 gilberts per cm. Three readings using these values were then made on each bar and the mean of these results were plotted as shown in Fig. 2. The lengths of the lines connecting the points indicate the spread of values of the induction B for F_F and the increment ΔB in induction for ΔF. The B points are indicated by the blocks and the ΔB points by circles.

A study of Fig. 2 shows that there is an overlap in both the B and ΔB values for bars that had received no draw and which had been quenched at 2316, 2366, and 2419° F. (1268, 1296, and 1326° C.), respectively, and there is also considerable overlap at the 1000 and 1050° F. (538 and 566° C.) drawing temperatures and 2316 and 2366° F. (1268 and 1296° C.) quenching temperature.

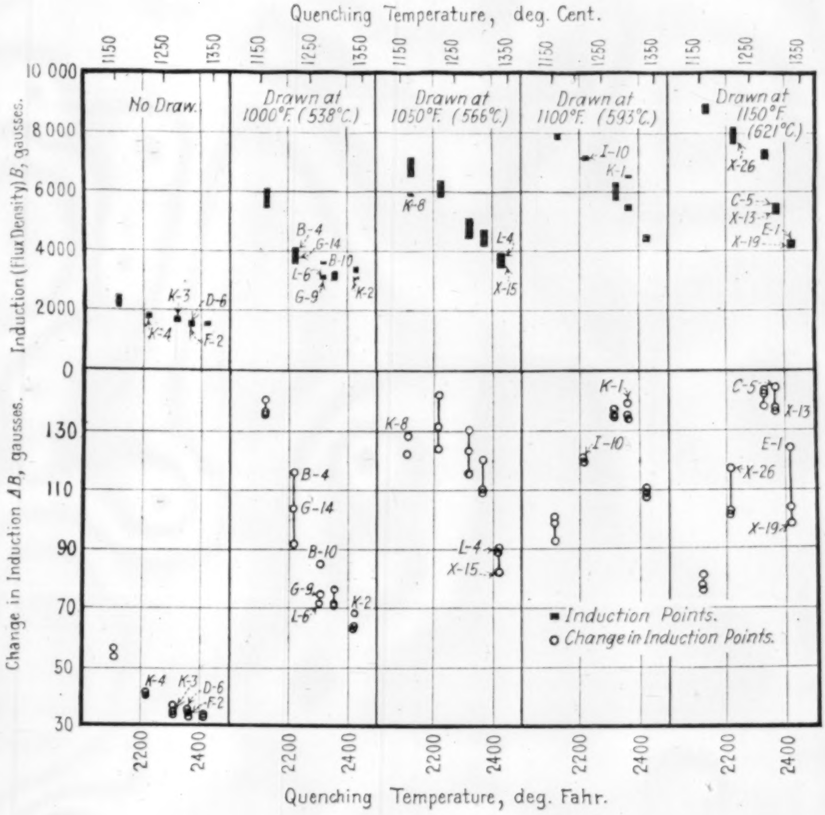

FIG. 2.—Results of Positive Incremental Test.

It is interesting to note that at the value of ΔF used in this test, the resulting increment in induction, ΔB, is to a large extent independent of the chemical composition of the specimens as shown by the position of the K bars. This, however, does not hold true at other values of ΔF.

NEGATIVE INCREMENTAL METHOD

In this method the same general procedure was followed as for the positive incremental method, except that ΔF was made negative and was larger than the constant F_F. The best values of F_F and ΔF in this case were found to be 106 gilberts per cm. and −138 gilberts per cm., respectively. The investigation covered a range in F_F from 50 to 150 gilberts and in ΔF from −4 to −150 gilberts per cm.

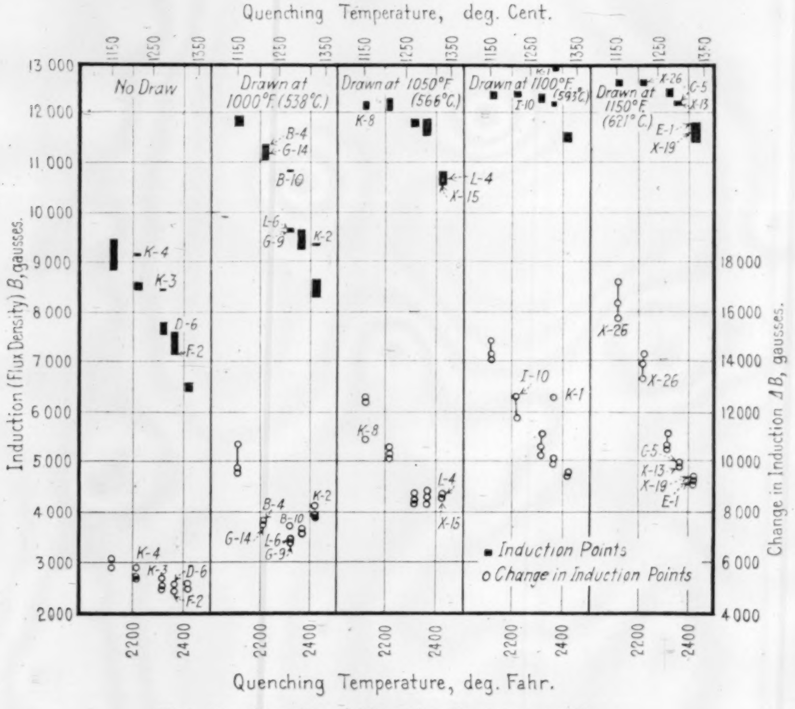

FIG. 3.—Results of Negative Incremental Test.

Three readings were taken on each bar with the above values of F_F and ΔF, and the agreement between individual readings is much better than in the positive incremental test. The mean values of B and ΔB for each specimen are plotted in Fig. 3. Comparing this with Fig. 2 it can be seen that at the higher values of induction used in this test there is less spread between the individual specimens of a given heat treatment. The K bars, however, do not line up with the others of the same heat treatment.

A study of Fig. 3 shows that the overlapping still occurs to some extent for specimens that were quenched at 2316 and 2366° F. (1268 and 1296° C.) and drawn at 1050° F. (566° C.) but that the separation of the other heat treatments is good.

ALTERNATING CURRENT INCREMENTAL METHOD

Solenoid Test:

It has been shown in previous investigations (1) that alternating current magnetic tests are easier to apply than direct current tests, and are in general quicker. They also possess the advantage that the results are more consistent.

In the alternating current incremental test using a solenoid, measurements were made of the power loss in the specimens produced by a superimposed alternating magnetomotive force. Measurements of the induced voltage in the secondary winding set up by the alternating ΔF were made on a few specimens, but the results were not promising. Therefore the voltage test was abandoned.

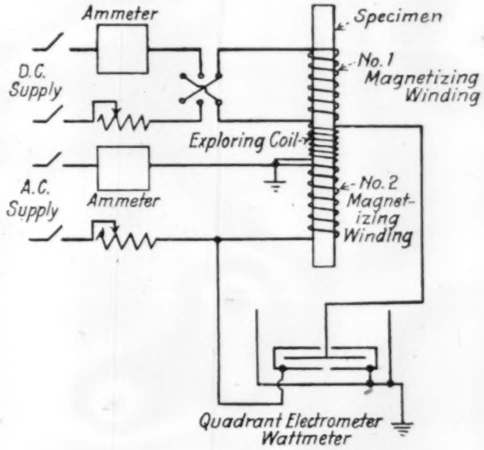
Ammeter
Specimen
D.C. Supply
No. 1 Magnetizing Winding
Exploring Coil
A.C. Supply
Ammeter
No. 2 Magnetizing Winding
Quadrant Electrometer Wattmeter

FIG. 4.—Connections for Alternating Current Solenoid Tests.

A quadrant electrometer, that was available, was used to measure the loss in the specimens. The connections for the test are shown in Fig. 4. The constant magnetomotive force, F_F, was supplied from a direct current source to the No. 1 winding of the solenoid and the alternating incremental magnetomotive force, ΔF, was applied to the No. 2 winding. The voltage circuit of the electrometer was connected to the secondary or exploring winding of the solenoid and the current circuit was supplied from No. 2 magnetizing winding. Although the readings of the electrometer with these connections are not the true power loss in the specimens, we have called the quantity measured the power loss. With these connections the I^2R loss in the solenoid causes a deflection of less than one centimeter; therefore the loss in the solenoid had a negligible effect upon the results

Test Procedure:

Two bars, *I-10* and *X-26*, both quenched at 2211° F. (1211° C.) and drawn at 1100° F. (593° C.) and 1150° F. (621° C.), respectively, were chosen for the preliminary work. The direct current incremental tests, see Figs. 2 and 3, served to separate these two bars fairly well, and they were selected in order to see if the separation could be increased by the use of the alternating current incremental test.

Tests were made in which the value of the constant magnetomotive force, F_F, was varied from 250 to 2500 ampere turns and the alternating magnetomotive force, ΔF, from 250 to 875 ampere turns. Starting with a demagnetized bar each time, F_F was applied and the iron brought into a cyclic condition. Then the alternating ΔF was applied and the reading of the wattmeter noted. The maximum separation between the two bars was found to take place under the following conditions:

1. A constant F_F of 1500 ampere-turns was applied and the specimen brought into a cyclic condition.

2. The constant magnetomotive force, F_F, was then reduced to 750 ampere-turns.

3. Then the alternating incremental force, ΔF, of 700 ampere-turns was applied and the power loss measured.

These preliminary tests on bars *I-10* and *X-26* were made at a frequency of 20 cycles. In the tests on the specimens the above procedure was also used at frequencies ranging from 14 to 30 cycles.

The phase angle of the voltage applied to the electrometer needle was also varied in an attempt to increase the separation between specimens of different heat treatments but without success. The widest separation was obtained with the connections shown in Fig. 4 and using the procedure outlined above.

Before making the alternating current tests, the temperature rise produced by an alternating magnetomotive force acting on a specimen was studied and also the effect of the length upon the loss in the specimens.

Temperature Rise:

The hysteresis and eddy current losses in the specimens under alternating magnetizing forces produce a rise in temperature. In order to determine the temperature rise that would occur, a specimen, *X-9*, which had been quenched at 2419° F. (1326° C.) and which had not been drawn, was chosen. This specimen was found by Sanford (6) to have the maximum coercive force of any of the bars tested It was placed in a 3-in. solenoid and subjected to an alternating magnetomotive force of 7500 ampere-turns at 20 cycles. The temperature rise of the specimen was measured by a thermometer and found to equal 20° C. (68° F.) above room temperature. It required over 2 hours for the specimen to reach a constant temperature. In the alternating current incremental test a specimen was never in the solenoid for more than a few minutes and any change in loss that took place due to heating was negligible. This statement was verified later when it was found that wattmeter readings did not change even if the specimen was left for several minutes in the solenoid.

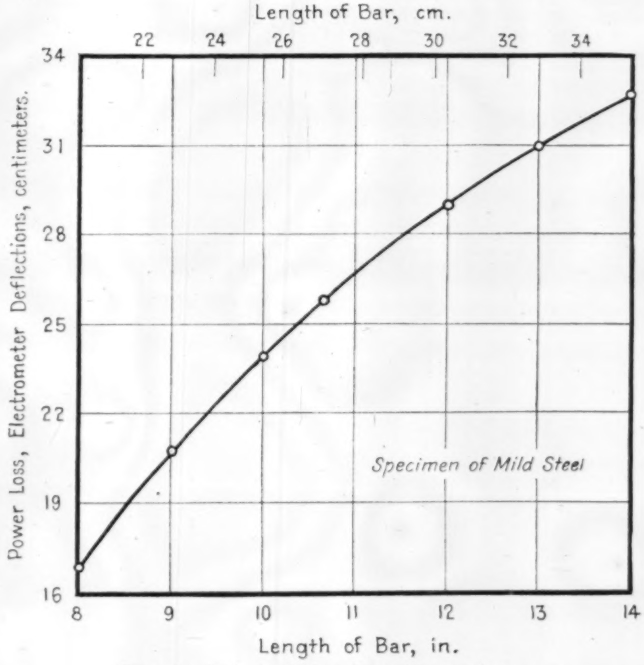

FIG. 5.—Effect of Length on Power Loss.

Effect of Length of Bars:

The total loss in a specimen of iron under an alternating magnetomotive force is proportional to the volume, hence it was necessary to determine the effect of the lengths of the bars on the loss readings of the wattmeter.

In the alternating current incremental tests the frequency of the alternating ΔF was varied from 14 to 30 cycles. No alternating current potentiometer was available for this range of frequency and therefore it was impossible to measure a component of the induced voltage which would be proportional to the loss as was done by Spooner (2) in his 60-cycle tests.

A bar of mild steel 14 in. (35.56 cm.) long was placed in the solenoid and the loss measured with an alternating current of 1.8 amperes at 20 cycles frequency flowing through the No. 2 winding of the solenoid. The bar was then removed and 1 in. (2.54 cm.) cut from its length and the loss again measured. This procedure was repeated until the bar had been reduced to a length of 8 in. (19.32 cm.). The results of this test are shown in Fig. 5, and as was to be expected the effect of length is considerable.

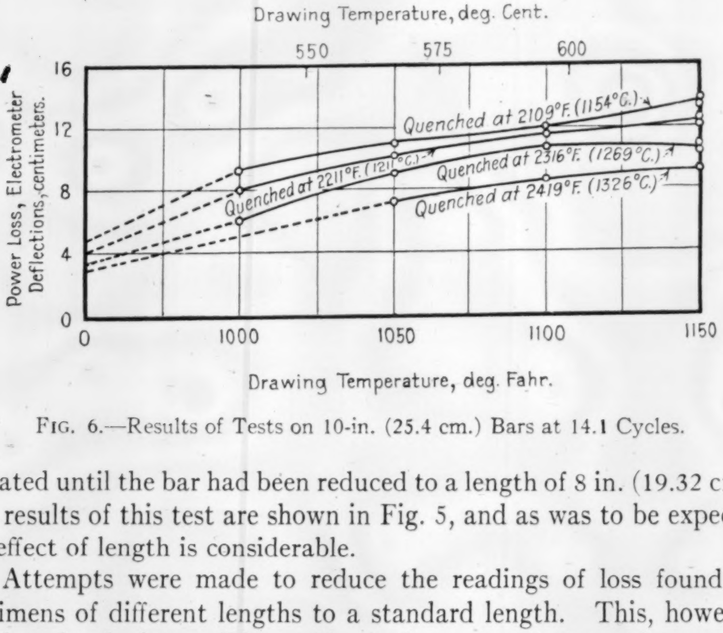

Fig. 6.—Results of Tests on 10-in. (25.4 cm.) Bars at 14.1 Cycles.

Attempts were made to reduce the readings of loss found for specimens of different lengths to a standard length. This, however, was found to be impossible, and in the incremental alternating current tests the bars were grouped according to their lengths. Thirty of the specimens were nearly exactly 10 in. (25.4 cm.) in length and these were used for the tests. These thirty specimens unfortunately only included two that had been quenched at 2366° F. (1296° C.) and therefore it was not feasible to plot curves for this quench.

Results:

Using the method described above, power loss measurements were made on the 10-in. bars at frequencies of 14.1, 20, and 24.5 cycles, the results of which are plotted in Figs. 6, 7, and 8. Not

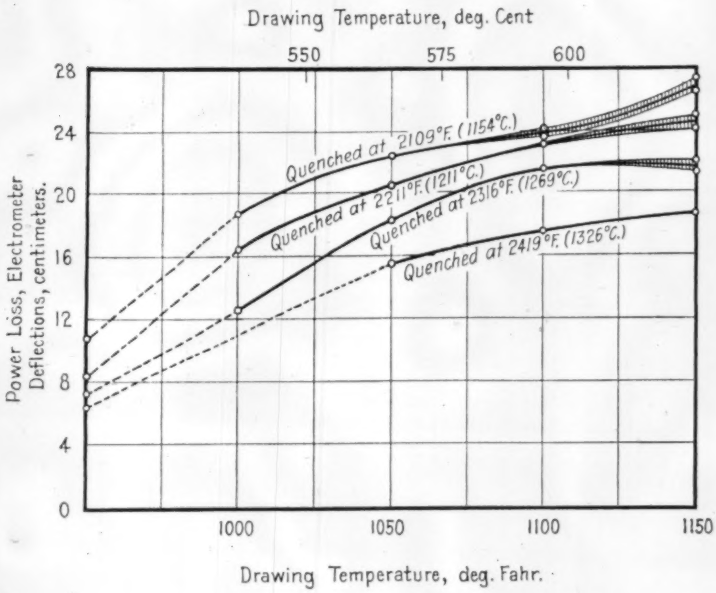

FIG. 7.—Results of Tests on 10-in. (25.4 cm.) Bars at 20 Cycles.

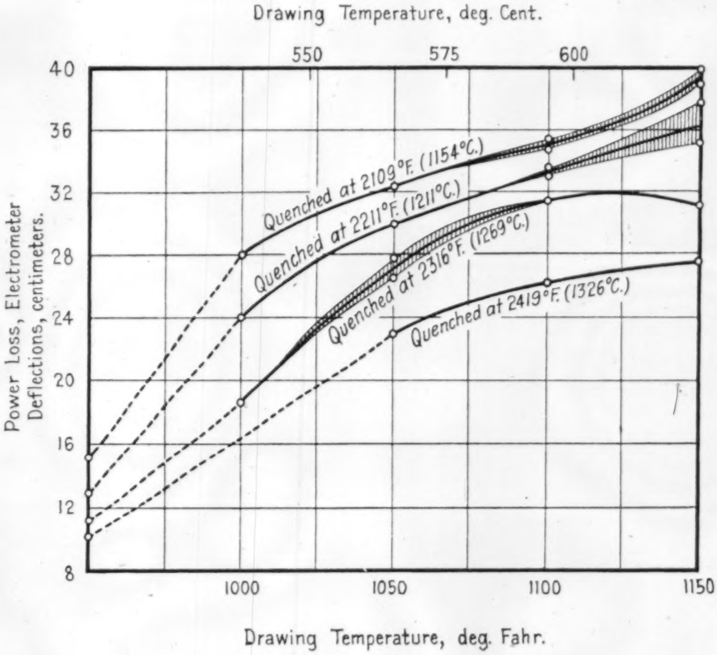

FIG. 8.—Results of Tests on 10-in. (25.4 cm.) Bars at 24.5 Cycles.

more than two bars of any one heat treatment were available in this group, and in most cases only one. The center lines of the cross-hatched areas represent the mean values of the measurements.

These curves are similar in form to those obtained by Spooner (2) in his differential tester with the alternating current potentiometer at

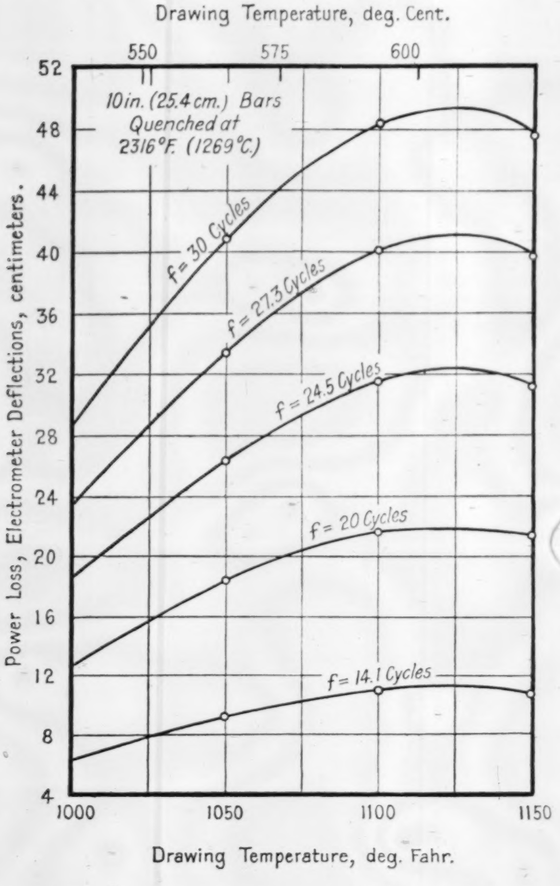

Fig. 9.—Effect of Frequency on Power Loss.

60 cycles frequency. At 14.1 cycles, Fig. 6, the separation between the different quenching temperatures is good, but the curves for any one temperature are quite flat. At 20 cycles, Fig. 7, the separation between the different quenching temperatures is better and the curves are steeper. At 24.5 cycles, Fig. 8, the separation is still better and the curves are steeper than in the two previous figures.

The tendency of the curves for the bars quenched at 2316° F. (1268° C.) to bend over at the higher drawing temperatures may be caused by secondary hardening as pointed out by Spooner (2). A further test was made on these bars at 27.3 and 30 cycles. The results of the power loss measurements on these 2316° F. (1268° C.) bars for the range of frequencies are plotted in Fig. 9. The bending over becomes greater as the frequency is raised.

The second largest group of bars of the same length was that of approximately 9.5 in. (24.2 cm.) which, unfortunately, were of too small a number to give representative curves as were obtained for the 10 in. (25.4 cm.) bars. The power loss in these specimens was measured under the same frequencies and test conditions as described for the 10-in. (25.4 cm.) specimens. Results obtained for these bars were similar to those found for the 10 in. (25.4 cm.) bars.

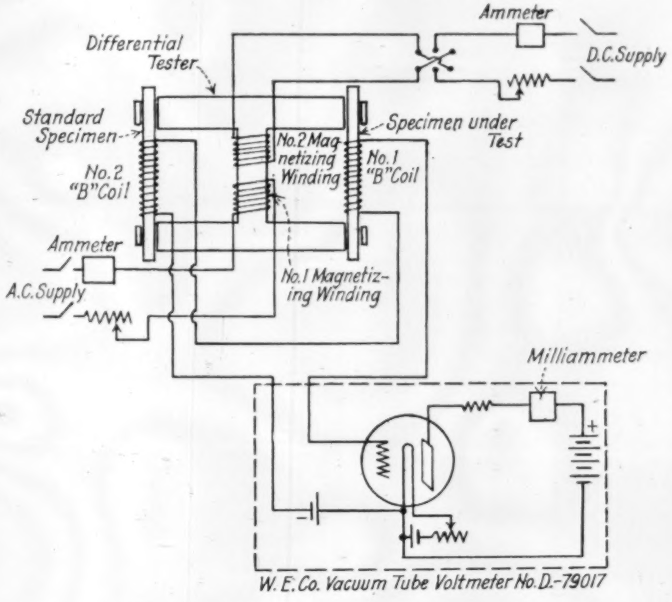
Ammeter
D.C.Supply
Differential Tester
Standard Specimen
Specimen under Test
No.2 Magnetizing Winding
No.1 "B" Coil
No.2 "B" Coil
Ammeter
A.C.Supply
No.1 Magnetizing Winding
Milliammeter
+
W. E. Co. Vacuum Tube Voltmeter No. D.-79017

FIG. 10.—Connections for Alternating Current Differential Test.

DIFFERENTIAL ALTERNATING CURRENT INCREMENTAL TEST

In the differential alternating current incremental test, using the special differential magnetic tester, a constant magnetomotive force F_F was applied to winding No. 2, and the alternating current incremental magnetomotive force to the No. 1 winding as shown in Fig. 10. Specimens *I-1* and *X-12*, quenched at 2366° F. (1268° C.) and drawn at 1100° F. (593° C.) were chosen as the standards and the other bars were referred to them. These bars were chosen as the standards because it had been brought out in previous work (3,8,10) that this combination of quenching and drawing temperatures represents the best heat treatment for cutting efficiency in the 18-per-cent tungsten steels. Bar *I-1* was placed in one of the "*B*" coils of the tester and the other bars were placed in turn in the other "*B*" coil, the coils being connected differentially. The difference in voltage induced in the two "*B*" coils under the influence of the alternating current magnetomotive force was measured by a Western Electric Co. vacuum tube voltmeter Type D–79017.

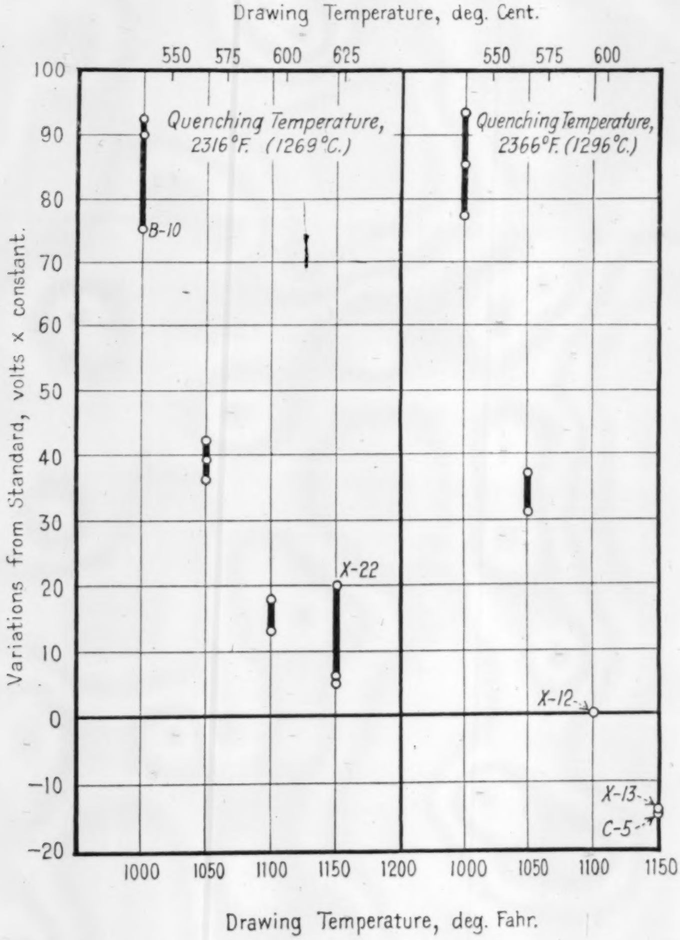

FIG. 11.—Results of Alternating Current Differential Test.

The purpose of this test was mainly to separate bars that had not received the standard heat treatment from those that had.

Test Procedure:

With the standard, bar *I-1* in No. 2 "*B*" coil and the bar under test in No. 1 "*B*" coil, the constant F_F was applied and reversed 12 times to put the bar in cyclic condition. The alternating incremental magnetomotive force was then gradually applied and brought up to the desired value. This procedure was found necessary in order to repeat results. The difference in the voltages induced in the two "*B*" coils by the alternating flux in the specimens was then read.

TABLE II.—RESISTIVITY OF SPECIMENS.

Bar	Drawn at		Resistivity, michroms per cu. cm.	Bar	Drawn at		Resistivity, michroms. per cu. cm.
	deg. Fahr.	deg. Cent.			deg. Fahr.	deg. Cent.	
Bars Quenched at 2109° F. (1154° C.)				Bars Quenched at 2366° F. (1296° C.)			
A-1	[a]	[a]	64.0	A-4	[a]	[a]	73.4
L-11	1000	538	56.7	L-1	1000	538	61.0
G-5	1000	538	50.3	X-6	1050	566	59.0
K-8	1050	566	48.5	H-4	1050	566	57.3
X-5	1050	566	52.7	X-12	1100	593	57.1
X-23	1100	593	53.8	K-1	1100	593	50.7
I-11	1100	593	50.4	X-13	1150	621	57.1
J-14	1150	621	48.4				
X-24	1150	621	41.6				
Bars Quenched at 2211° F. (1211° C).				Bars Quenched at 2419° F. (1326° C.)			
K-4	[a]	[a]	61.7	X-9	[a]	[a]	74.8
L-7	1000	538	57.4	X-11	1000	538	59.4
X-14	1050	566	55.7	K-2	1000	538	57.3
H-10	1050	566	55.8	X-15	1050	566	60.7
X-20	1100	593	55.1	G-2	1050	566	58.1
X-26	1150	621	53.1	X-2	1100	593	56.1
				X-7	1150	621	54.5
Bars Quenched at 2316° F. (1268° C.)							
K-3	[a]	[a]	65.6	X-16	1100	593	54.1
L-6	1000	538	57.3	X-22	1150	621	52.4
X-14	1050	566	57.7	J-6	1150	621	52.2
C-9	1050	566	56.2				

[a] No draw.

The value of the constant magnetomotive force F_F, was chosen to give a flux density of about 10,000 gausses in the standard specimen. The alternating incremental magnetomotive force used was of such a value as to give an incremental magnetizing force of approxiamtely 20 gilberts per centimeter. Under these conditions there was no appreciable rise in temperature of either specimen. The voltmeter indication obtained with bar *X-12* in the tester was taken as the zero of the instrument and the other bars referred to this.

Results:

Tests were made on those bars which had been quenched at 2316° F. (1268° C.) and 2366° F. (1296° C.), as these bars had shown little separation in the other tests previously described. Measurements were made with alternating incremental magnetomotive forces at 20, 40, 60, and 133 cycles frequency. The best separation occurred at 60 cycles. At 20 and 40 cycles there was no separation and only slight separation at 133 cycles. The results of the tests at 60 cycles are plotted in Fig. 11. A study of this figure shows that there is no overlap between the standard specimens and any of the other specimens tested.

RESISTANCE MEASUREMENTS

The resistances of some of the bars were measured in a Kelvin Double Bridge and the results are given in Table II. Some of the data given in this table were furnished by Mr. Sanford of the U. S. Bureau of Standards. The measurements were made at 68° F. (20° C.).

A study of this table shows that the *K* specimens are in general out of line with others of the same heat treatment; that the check between two bars having the same heat treatment is in the main good; and that at any one quenching temperature there is a decrease in resistance with increase in drawing temperature.

CONCLUSIONS

Erratic Bars:

As stated before, a number of investigators have tested these bars and found some erratic ones, whose magnetic properties do not agree with other bars of supposedly the same heat treatment.

In this investigation the magnetic properties of specimen *B-10* differs widely from bars *L-6* and *G-9* which were supposed to have had the same heat treatment. This confirms de Forest(4) results on the same bars. Bars *L-6* and *G-9* appear in the direction of a higher quench.

The results of the incremental magnetic tests on the other hand show but little difference between the following pairs of bars: *D-6* and *F-2*; *X-15* and *L-4*; *X-13* and *C-5*; and *G-14* and *B-4*, which were reported out of line by some of the other observers(3,4,8).

The magnetic properties of specimen *E-1* as found do not check in the positive incremental test with other bars of the same heat treatment. This result agrees with the results of Spooner(8) and de Forest(4). In the negative incremental test, however, this bar appears to be properly heat treated.

The different results obtained by the various investigators upon the same bar depends to some extent upon the values of induction at which the tests were made.

Summary:

In this investigation the best results were obtained at high inductions, the separation between the bars being greater in the direct current negative incremental test where a high induction was used than in the positive incremental test. This result also confirms the data obtained by other investigators(2).

If the quenching temperature is known, the drawing temperature may be determined fairly accurately by the loss measurement using the solenoid in the alternating current incremental test, except in the case of specimens quenched at 2316° F. (1268° C.), in which case the curve bends over. This test will not differentiate between specimens which have been quenched at 2316° F. (1268° C.) and drawn at 1100° F. (593° C.) and 1150° F. (621° C.).

If either the quenching or drawing temperature is known, the direct current negative incremental test serves to separate the bars fairly well; better than the alternating current solenoid test. Furthermore, if neither the quenching nor drawing temperatures are known, the direct current incremental tests offer a simple and reliable method of locating the specimens according to their heat treatments.

By means of the differential magnetic tester and an incremental alternating current test it is possible to distinguish between the bars that are correctly heat treated and those that are not.

The values of the constant magnetizing force and the incremental magnetizing force used in this investigation apparently are the best for the specimens tested. However, further investigation may develop different combinations of magnetizing forces which may produce better results.

In this work the specimens had two variables: namely, quenching and drawing temperatures; therefore two magnetic characteristics are necessary for differentiating between them. An incremental magnetic test offers a simple means of obtaining the needed magnetic properties.

The incremental tests are easy to apply, quick and reproducible. They offer a new tool for use in magnetic analysis.

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(3) A. V. de Forest, Discussion on Magnetic Analysis, *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 155 (1926).

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ENDURANCE OF HIGH-SPEED CUT-OFF TOOLS IN RELATION TO MAGNETIC AND OTHER MEASUREMENTS.

By Haakon Styri[1]

Synopsis

A number of standard cut-off tools hardened in groups from different temperatures have been used in cutting off rings of high-carbon chromium steel having a Brinell hardness of about 180. The number of rings have been counted for each grind. At least five grinds have been made on each tool and the average number of rings cut per grind calculated.

Magnetic and electric tests, and Rockwell hardness and duroscope readings were made on these tools, and the relation between these values and the number of rings cut per grind are shown.

The methods of measurement are described and possible reasons for irregularities in results are indicated.

The tools used in this investigation were cut from two bars of the same heat of high-speed steel of the 18-4-1 type with 0.70 per cent of carbon. The tools were heat treated by the manufacturer according to specified quenching temperatures. Four tools were quenched at each of the following temperatures: 2100, 2200, 2300, 2400 and 2500° F. (1150, 1200, 1260, 1315 and 1370° C.). All tools were drawn at 1050° F. (565° C.). The manufacturer stated that the temperatures were measured with an optical pyrometer. No record was kept of the bar from which the different tools were taken and the time the tools were held at quenching and drawing temperature was not recorded, the method of quenching being the standard commercial method. The tools for quenching were heated in an open oil-fired furnace. One of the tools had warped so badly that it was annealed and requenched. No great accuracy can, therefore, be claimed for the heat treatment.

A paper presented last year by A. V. deForest[2] gave oscillograms of these tools showing irregularity for tools Nos. 2 and 16, and in a paper presented last year,[3] J. A. Sams gave a table of Duroscope readings made by himself and Rockwell readings made by E. Dipper

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[1] Director of Research, S. K. F. Industries, Inc., Philadelphia, Pa.

[2] A. V de Forest, "A Method of Graphic Representation of Magnetic Characteristics," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 234 (1927).

[3] J. A. Sams, "The Duroscope: An Apparatus for the Magnetic Determination of the Durability of Steels," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 252 (1927).

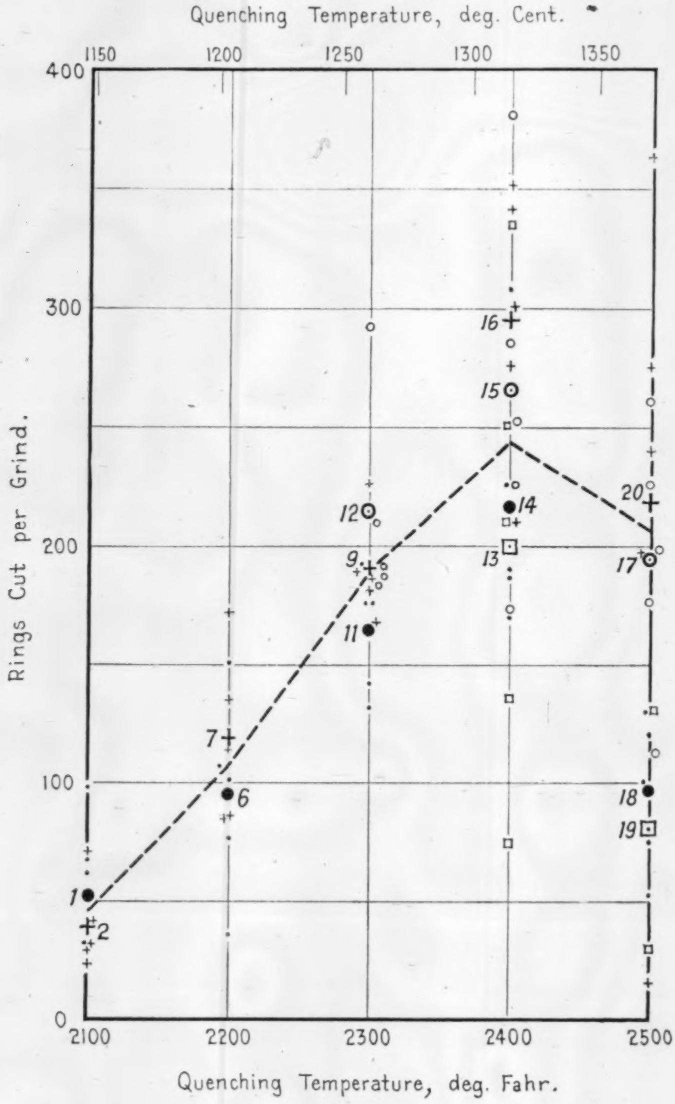

FIG. 1.—Relation Between Cutting Properties of Tools and Quenching Temperature.

of the S. K. F. Research Laboratory, indicating that tool No. 18 was different from the rest of the tools in the group and possibly is the tool referred to above as being requenched.

During the year shop tests have been made on these tools for comparison of endurance under operating conditions. The tests were run on chromium ball-bearing steel with about 1 per cent of carbon and 1½ per cent of chromium and with a Brinell hardness of about 180.

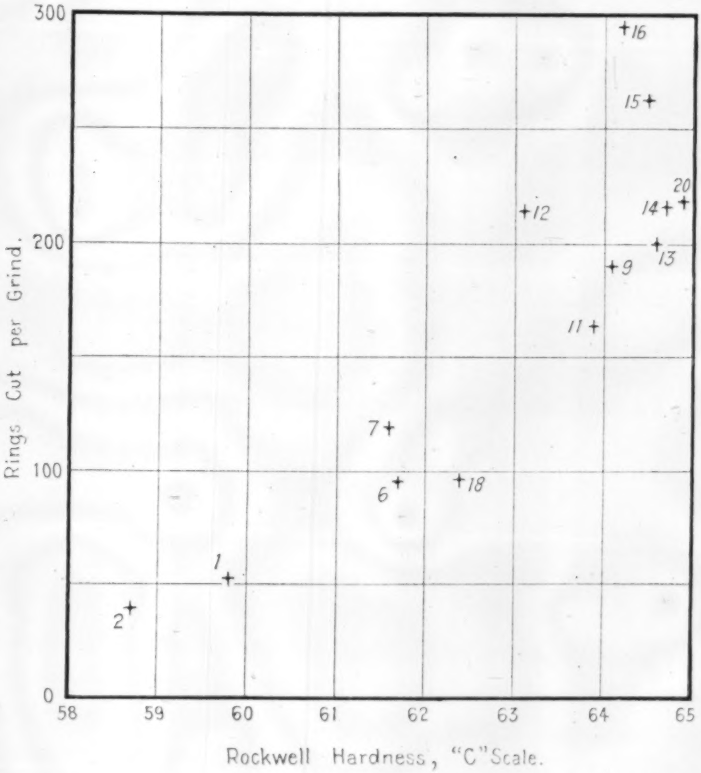

FIG. 2.—Relation Between Cutting Properties of Tools and Rockwell Hardness.

The tests were run under as similar conditions as possible. The tools were kept in operation until the edge got dull and the tool was then ground back sufficiently to give a clean cutting edge, usually about ¼ in. back from the previous edge. There was considerable variation in the number of rings cut per grind and it was found desirable to make five grinds in order to get a fair average. The individual number of rings cut for each grind as well as the average per grind of the five are given in Fig. 1. The average number of rings cut are also plotted against the Rockwell hardness of the tools in Fig. 2.

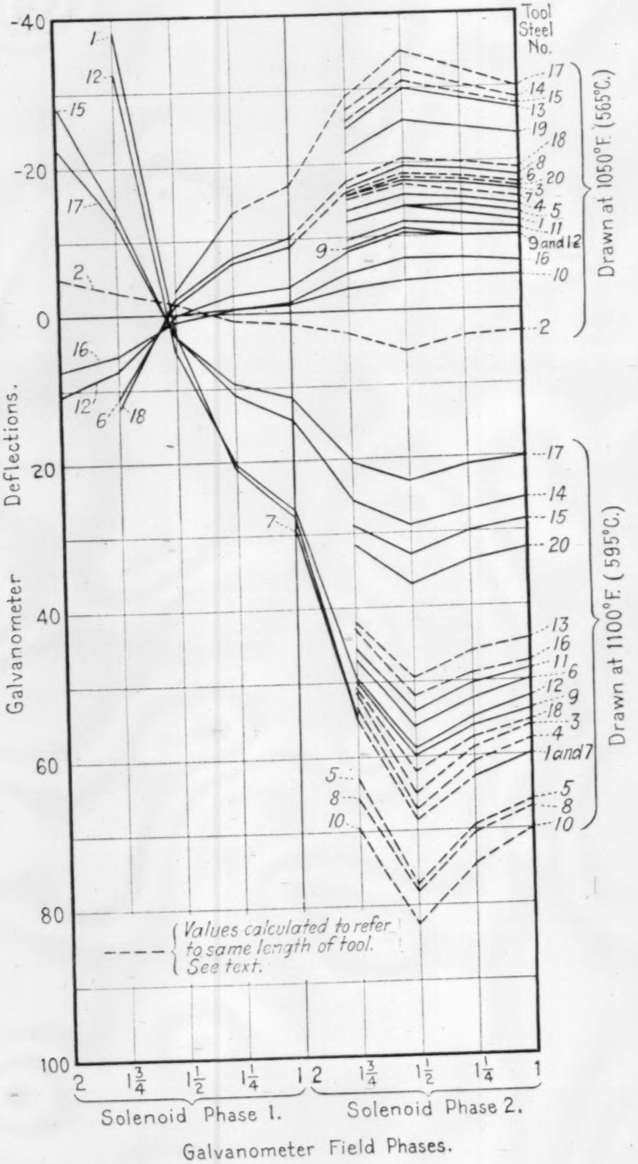

FIG. 3.—Magnetic Analysis of Tools Quenched at Various Temperatures and Drawn at 1050 and 1100° F.

Refer to Fig. 1 to identify tool numbers with corresponding quenching temperatures.

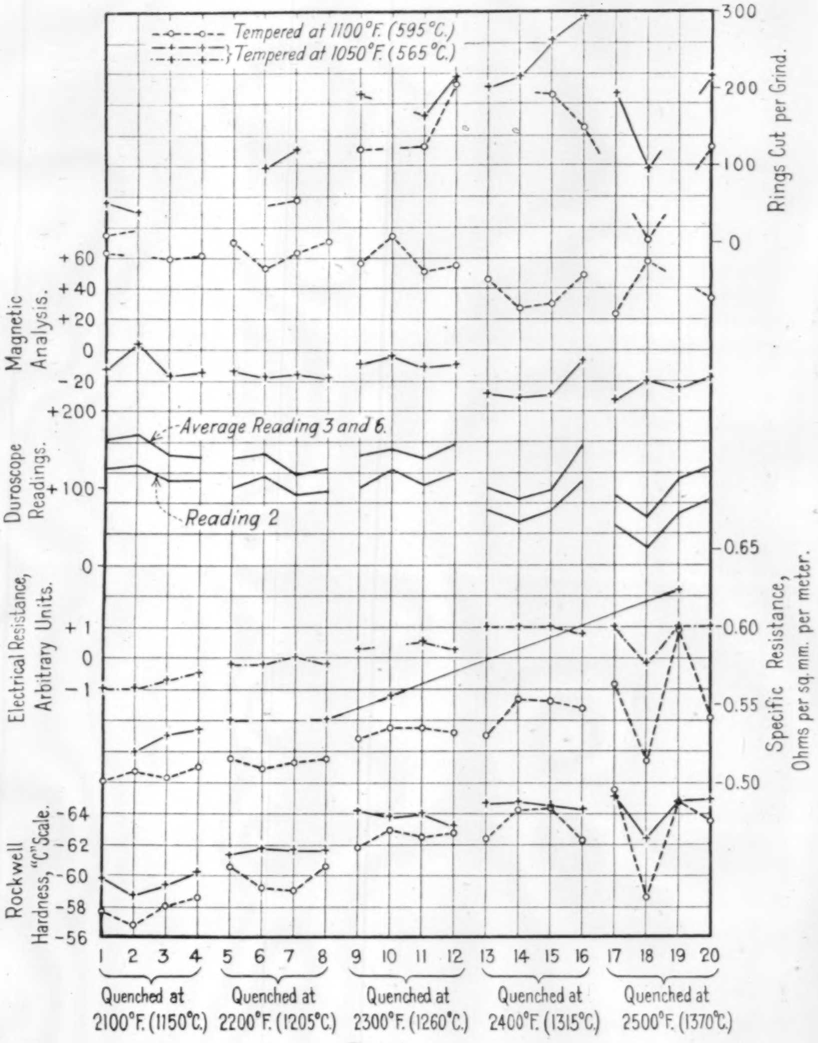

FIG. 4.—Variation of Magnetic, Electrical and Physical Properties with Quenching Temperatures of Tools.

It was at first not intended to run more than a couple of tools in each group and for that reason not all the tools were compared magnetically in the original condition. The magnetic tests were made according to the method described in a paper presented before the Society in 1926,[1] with the exception that the phase shifting was done by connecting the terminals of the two-phase system with an inductive resistance and selecting intermediate points on this resistance for the phase shifting instead of using condensers. It was later found that it was desirable to run tests on more of the tools, particularly in the groups quenched at 2300 and 2400° F. (1260 and 1315° C.), and it was therefore necessary to compare magnetic tests on the shorter used tools with tests on the longer unused tools. This was done by comparing the effect of length of tool on the magnitude of the magnetic reading and making corrections so that the readings represented in Fig. 3 refer to the same length of tools. The dashed lines in the figure show calculated values of readings for the tools for which direct readings were not made.

The values of the magnetic readings taken with the phase setting giving most of the readings directly on the galvanometer scale as reference point (phase two in solenoid and $1\frac{1}{4}$ in. field), and the number of rings per grind, the average of five tests, are given in Fig. 4. In the graph the other physical data have also been plotted against each tool and quenching temperature, the Rockwell readings as previously given by Mr. Dipper, the Duroscope reading No. 2 and the average of Duroscope readings Nos. 3 and 6 taken from Mr. Sams's paper, and the electrical resistance measured in our laboratory by a differential method. The electric resistance measurements, which are given in arbitrary units (the dot-and-dash line), cannot be considered very accurate, and in order to give an idea of the actual magnitude of this resistance, more accurate readings made by means of a precision potentiometer are added. All the other measurements made on tools drawn at 1050° F. (565° C.) are shown in full lines.

It will be seen from Figs. 1, 2 and 4 that there is a general correlation between Rockwell hardness, electric resistance and number of rings cut per grind up to 2400° F. (1315° C.) quenching temperature, but that the relation does not hold so well for 2500° F. (1370° C.) quenching. It is interesting to note that tool No. 18, which gave very low Rockwell hardness and electrical resistance, also gives poor endurance in cutting. There is no such general relation between the magnetic readings and the other physical tests except the Duroscope, and

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[1] Haakon Styri, "Testing of Ball Bearing Races by Electric and Magnetic Methods," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 152 (1926).

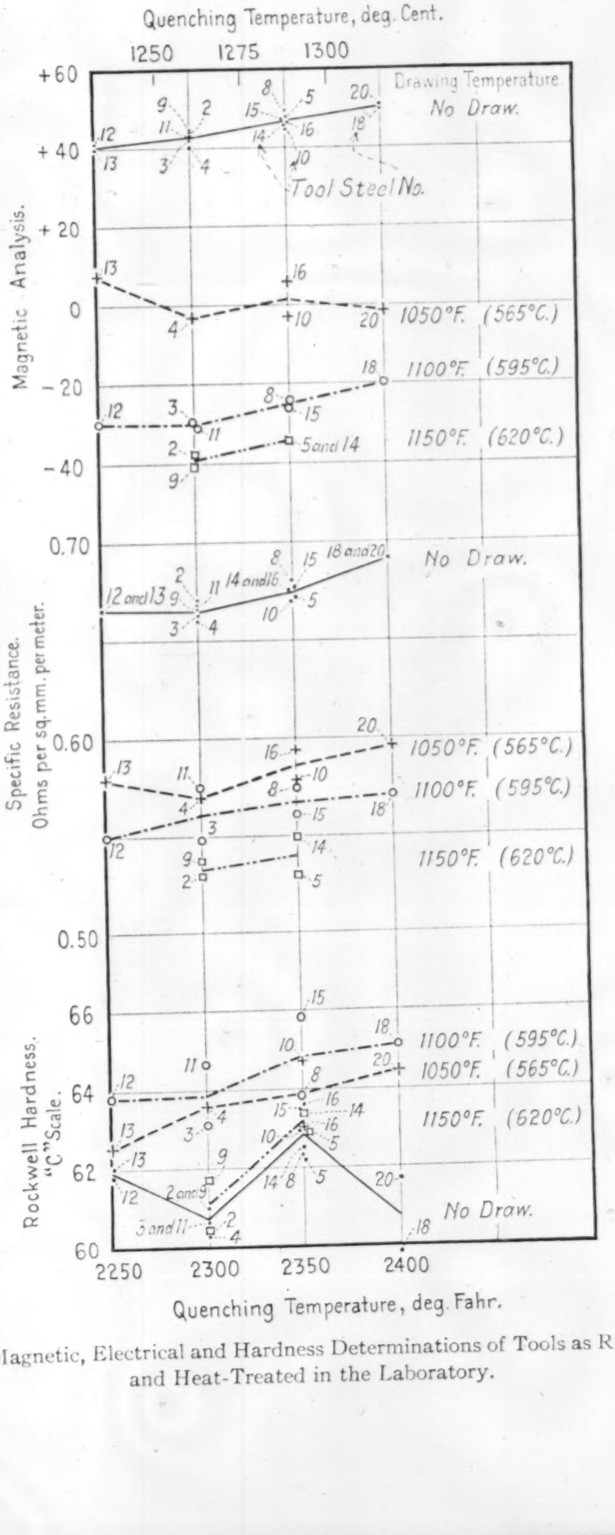

FIG. 5.—Magnetic, Electrical and Hardness Determinations of Tools as Re-Annealed and Heat-Treated in the Laboratory.

here it is of interest that there seems to be a reversal in average values at about 2300° F. (1260° C.) quench. Within each group of the same quenching temperature there is, however, some relation between magnetic readings, Duroscope readings and the other physical data, although not at all consistent. Tools Nos. 2 and 16 which gave different oscillograms deviate noticeably.

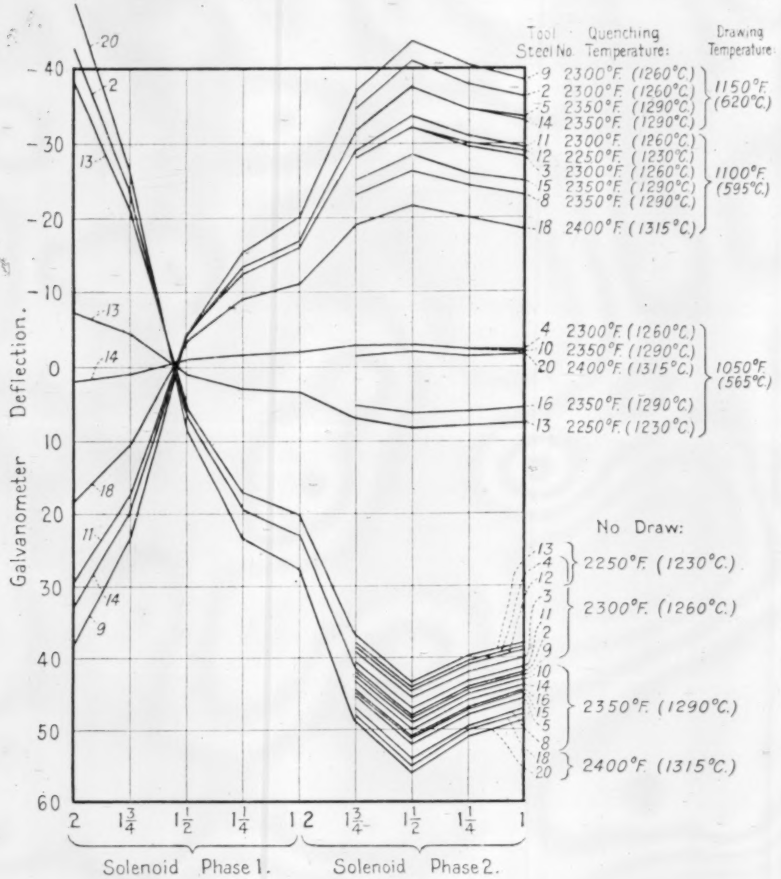

FIG. 6.—Magnetic Analysis of Tools as Re-Annealed and Heat-Treated in the Laboratory.

It was suspected that perhaps variations in heat treatment had caused this inconsistency, as it is known that drawing temperature has a very great influence on magnetic readings. The tools were therefore redrawn at 1100° F. (595° C.) and a complete set of new measurements (except Duroscope readings) and even a few cut-off tests were made; however, with only a couple of grinds for each tool. These new readings are shown in Fig. 4 with dashed lines. It is shown that Rockwell hardness in most cases dropped, as did the electrical resistance. The greatest influence is shown on the magnetic reading, where evidently the drawing temperature has very much greater influence than differences in hardening temperature. The number of rings cut per grind has also dropped. The magnetic reading still shows unsatisfactory correlation with the other readings. As this did not seem to check with previous investigations on other types of material, it was decided to reanneal the tools and to heat-treat them in the laboratory.

They were all annealed at 1400° F. (760° C.) and quenched as follows: Nos. 12 and 13 from 2250° F. (1230° C.), 3 minutes in furnace; Nos. 2, 3, 4, 9 and 11 from 2300° F. (1260° C.), 2.5 minutes; Nos. 5, 8, 10, 14, 15 and 16 from 2350° F. (1290° C.), 2 minutes; and Nos. 17 and 20 from 2400° F. (1315° C.), 2 minutes in furnace; all from a Globar furnace after pre-heating in another furnace. The temperature was measured with platinum-platinum-rhodium couple. The tools were further divided in three drawing groups as shown in Fig. 5, drawn in a salt bath. The magnetic readings for these tools are shown in Fig. 6.

In this case it is interesting to note that there is direct correlation between electrical resistance and the magnetic analysis, but it will also be noticed that there is very little difference in magnetic readings for differences in quenching temperature and very great difference for variations in drawing temperature. With particular reference to drawing at 1050° F. (565° C.), it so happened that tools Nos. 4, 10 and 20 were drawn at one time and Nos. 13 and 16 drawn at another, and evidently the drawing conditions have not been kept exactly alike. The tools drawn at 1100 and 1150° F. (595 and 620° C.) have, however, been drawn simultaneously for the same group. There is a continuous drop in the electrical resistance with increase in drawing temperature. The increase in Rockwell hardness shows a maximum at about 1100° F. (595° C.) for each of the quenching temperatures used.

CONCLUSION

From the results obtained in this investigation it would appear that the magnetic analysis does not have any relation to the endurance of the tool, but that on the other hand hardness and electrical resistance measurements give quite good correlation. The magnetic test, however, when performed for a known heat treatment very readily shows deviation from this heat treatment. Considering the great influence of the drawing temperature on the magnetic reading, it may be advisable under shop conditions to test tools magnetically after quenching and again after drawing, for uniformity. The test under standard conditions takes very little time and is not destructive; it is readily undertaken and can be made as sensitive as desired, but is necessarily as yet only comparative.

Acknowledgments.—This investigation was first started by Mr. E. Dipper at the S. K. F. Research Laboratory in cooperation with the Hess-Bright Manufacturing Co. of the S. K. F. Industries, but the author is particularly indebted to Reidar Goffeng, Fellow of the American-Scandinavian Foundation, for his careful work in the taking of the majority of these measurements and making the graphs.

DISCUSSION

Mr. A. V. de Forest.[1]—In regard to the diagram showing measurements at different phase relations shown in Fig. 6 of Mr. Styri's paper, the important part of that diagram, the place close to where the curves all cross, is not shown at a sufficient scale to be visible. If the sensitivity of that portion of the diagram is increased so as to get a good distribution in that region, it will be seen that there are two variables present, one of them affecting the distribution on the plot as shown by Mr. Styri and the other affecting the place at which those curves cross the zero line, and that a combination of these two magnetic properties can be made over a certain range to handle the two variables of quench and draw at the same time. Of course it is impossible to handle two variables with one measurement, but the two magnetic variables are there and could be made use of. **Mr. de Forest.**

I think, however, as Mr. Styri states, that the easiest way is to make two measurements, one after the quench and one after the draw. In that way a very much cruder measurement suffices to distinguish between the materials. If, however, that method is not available, as for instance if the tool is being investigated by the consumer, two measurements must be used in order to get the answer. I believe that whenever magnetic measurements are made that do not correlate with the mechanical properties which are known to be present, the magnetic measurements are not necessarily wrong in themselves, but are unwisely selected. The answer lies in selecting a measurement which will correspond to what we want to know.

Mr. Haakon Styri.[2]—I am glad Mr. de Forest called attention to the crossing point, because I was after that to quite some extent. We would have to increase the sensitivity of the galvanometer to get the reading between tools which had been quenched at different temperatures only, while tools which had been drawn at different temperatures required less sensitivity. With the same scale, the lines would necessarily come so close to each other near the crossing points that the sequence would not show in the graph, as there is too little variation in the heat treatment to show noticeable displacement of the crossing points. When we made an examination of bearing steel two years ago we had very much greater variation in the drawing **Mr. Styri.**

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[1] Research Engineer, American Chain Co., Inc., Bridgeport, Conn.

[2] Director of Research, S. K. F. Industries, Philadelphia, Pa.

Mr. Styri. temperatures, and at that time it was very easy to select readings in two different directions which as a rule would identify the heat treatment. However, even then, several readings would overlap because a higher draw would counteract a higher quenching temperature and give approximately similar readings in the two phases and I do not believe the method is reliable in general.

CONCERNING THE YIELD POINT IN TENSION

By J. M. Lessells[1]

Synopsis

This paper criticises the methods used in the determination of certain tension test values, which methods have been suggested by the Society for standardization. The paper shows that the "Johnson limit" may, in certain cases, be far removed from the proportional limit and suggests certain improvements in the yield point determination.

Elastic Limit or Proportional Limit

The elastic limit is usually defined as the maximum stress to which a material can be subjected without, on removal of stress, showing permanent deformation. The proportional limit is, on the other hand, that stress where the deformation ceases to be proportional to the applied stress. Engineers usually assume that these values are the same for metals used in construction. Although these values, requiring sensitive apparatus, are difficult to obtain in actual tests and although their actual existence is, in certain quarters, very much in doubt, they are still very important to the designer since they definitely mark the limit of the applicability of all formulas based on the elasticity of the materials used. In view of the fact that the above values are difficult to determine, certain approximate methods are being used. These methods, two in number, are fully described in the Society's Tentative Methods of Tension Testing of Metallic Materials (E 8 - 27 T)[2] and it is with Method II that we are concerned at present. This method, proposed by the late J. B. Johnson, gets over the difficulty of ascertaining the exact point where the stress-strain curve ceases to be proportional by drawing a tangent to the curve the slope of which tangent represents a 50 per cent greater rate of deformation than the initial rate. Referring to Fig. 1, from data furnished by Mr. Irwin, of our organization, these values are shown plotted for two different grades of steel castings. It will be noted from Fig. 1(*a*) that there is good agreement between the Johnson and proportional limit, but in Fig. 1(*b*) there is a considerable

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1074 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 732.

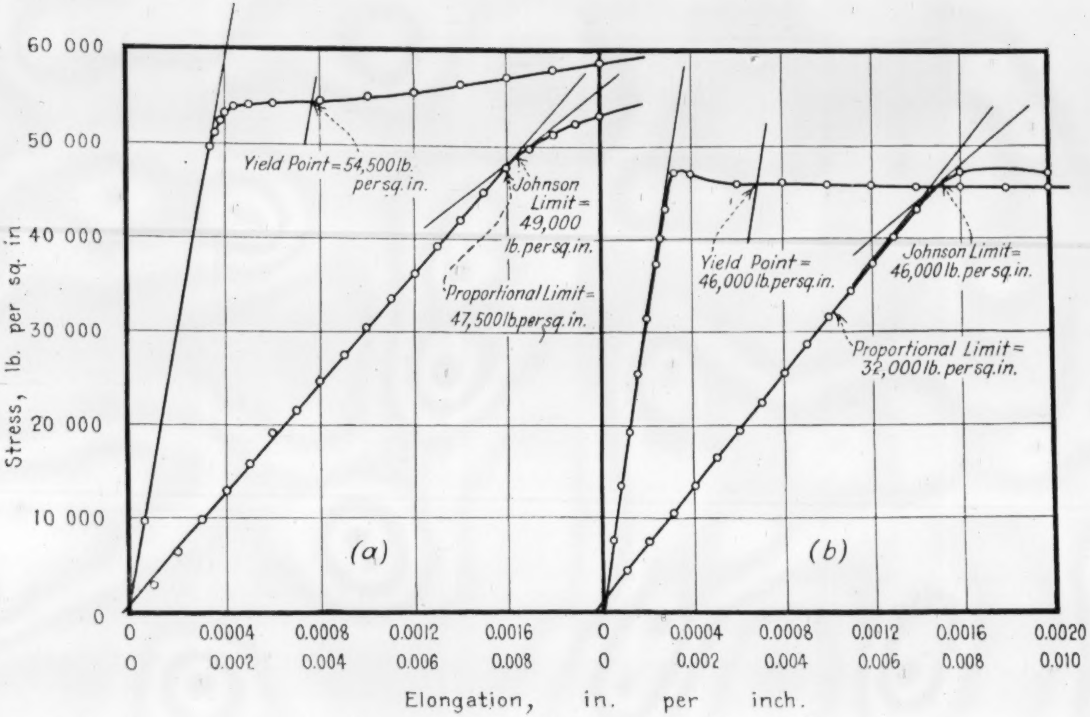

FIG. 1.—Tension Test Curves on Two Grades of Steel Castings.

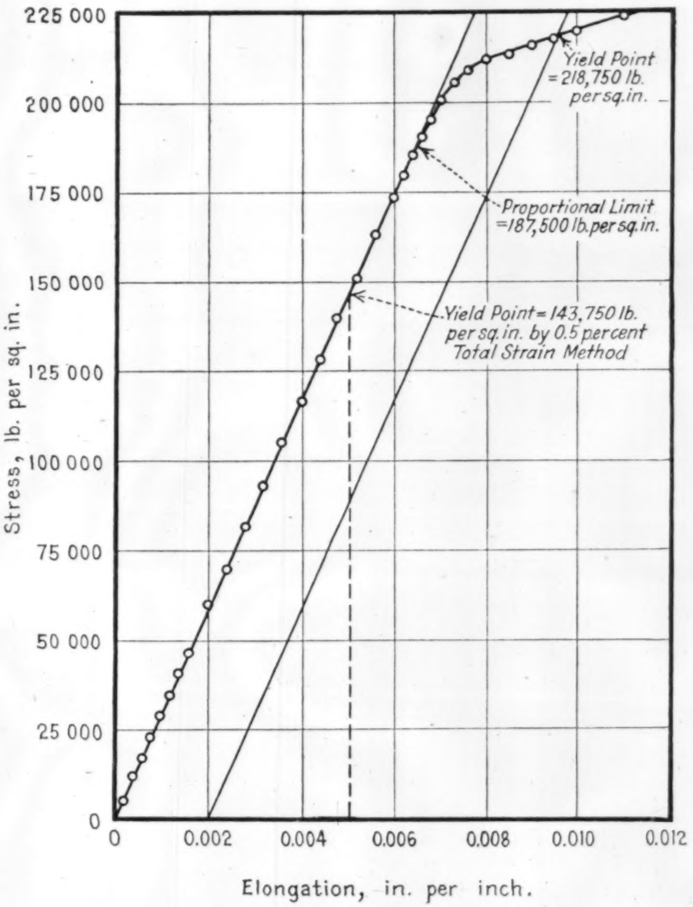

FIG. 2.—Tension Test Curve on Elastic Stage Chrome-Vanadium Spring Steel.

Proportional Limit, 187,500 lb. per sq. in.
Yield Point, 218,750 lb. per sq. in.
Tensile Strength, 245,200 lb. per sq. in.
Elongation in 2 in., 8.9 per cent.
Reduction of Area (0.505-in. diameter specimen), 34.5 per cent.
Modulus of Elasticity, 29,000,000 lb. per sq. in.
Brinell Hardness, 444.
Ewing Extensometer.
Amsler Hydraulic Machine.

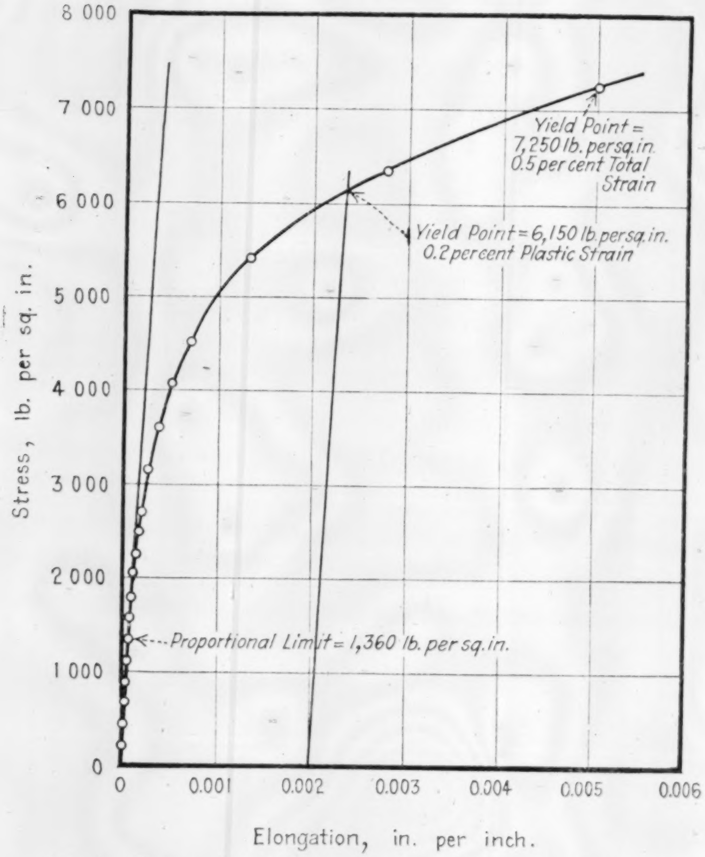

FIG. 3.—Tension Test Curve on Annealed Electrolytic Copper.

Proportional Limit, 1,360 lb. per sq. in.
Yield Point, 6,150 lb. per sq. in.
Tensile Strength, 31,300 lb. per sq. in.
Elongation in 1 in., 64.0 per cent.
Reduction of Area (0.375-in. diameter specimen), 67 per cent.
Modulus of Elasticity, 16,400,000 lb. per sq. in.
Brinell Hardness, 41.
Martens Extensometer.
Amsler Hydraulic Machine.

difference. If reference be made to a paper by D. J. McAdam, Jr.[1] and also one by N. L. Mochel,[2] it will be found that for the case of stainless iron far greater differences will be obtained. In the former paper a proportional limit of 25,000 lb. per sq. in. and a Johnson limit of 72,000 lb. per sq. in. are recorded. It is clear then from the above that where working stresses are being chosen these differences must be clearly understood; otherwise plastic deformations may be encountered in service.

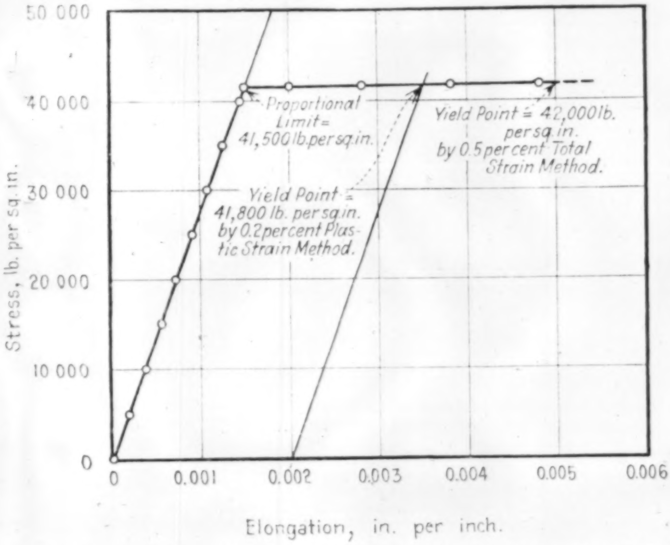

FIG. 4.—Tension Test Curve on 0.33 Carbon Steel Casting, Normalized.

Proportional Limit, 41,500 lb. per sq. in.
Yield Point, 41,800 lb. per sq. in.
Tensile Strength, 83,550 lb. per sq. in.
Elongation in 2 in., 29.1 per cent.
Reduction of Area (0.505-in. diameter specimen), 46 per cent.
Modulus of Elasticity, 28,000,000 lb. per sq. in.
Brinell Hardness, 163.
Ewing Extensometer.
Olsen Lever Machine.

YIELD POINT

Let us now consider the question of yield point. Referring to the Tentative Methods of Tension Testing of Metallic Materials (E 8 - 27 T)[3] this is defined as that stress at which a marked increase in strain occurs without an increase in stress. It may be mentioned that even where such a phenomenon does not exist it is customary to choose an arbitrary yield point as that stress corresponding to some definite extension. There are again two methods of determining the yield point, one by the drop of the beam of the testing machine and the second by the use of a strainometer, to determine a given extension. It is this second method with which we are concerned here.

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[1] D. J. McAdam, Jr., "Endurance Properties of Corrosion-Resistant Steels," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 273 (1924).

[2] N. L. Mochel, "Stainless Iron," *Transactions*, Am. Soc. Steel Treating, Vol. X, No. 3, p. 353 (1926).

[3] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1074 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 732.

In Method II of the Society's proposed standard, the yield point is taken as that stress corresponding to a total strain (elastic plus plastic) of 0.5 per cent. If reference be made, however, to Fig. 2, it will be observed that such a yield point for the case of a spring steel will fall *below* the proportional limit. Again from Fig. 3, for annealed copper, it will be seen that the yield point (by the 0.5-per-cent total strain method) is very far removed from the point representing cessation of proportionality. To surmount this difficulty the Westinghouse Electric and Manufacturing Co. has adopted a method shown in Figs. 2, 3, and 4 (the data again being supplied by Mr. Irwin) whereby the yield point is chosen on the basis of plastic strain—the amount being 0.2 per cent. This method is recommended for consideration by the Society. With such a method, the yield point will never fall below the proportional limit, will always represent a definite amount of plastic strain, and will always correspond to the value obtained by the drop of the beam if such exists.

CONCLUSIONS

In view of the present excellent work being done by the Society's Committee E-1 on Methods of Testing and also due to the fact that there are indications that certain steel specifications will be revised in the near future, it is thought that this is an opportune time to bring these points to the attention of the Society. With regard to Johnson limit, if the use of this value is continued, care should be taken to record it as such so that it will not be confused with the limit of proportionality. Moreover, since we have found serious disadvantages in the Society's tentative method of determining yield point, the 0.2-per-cent plastic strain method is hereby recommended.

DISCUSSION

Mr. J. A. Capp[1] (*presented in written form*).—Mr. Lessells' paper is both interesting and disappointing. It is interesting because it again demonstrates that for metals other than wrought iron and mild steels there is not apt to be found any definite clear-cut jog in an elastic curve to which the name "yield point" may justifiably be applied. It is disappointing because it then proposes another definition for something the non-existence of which it has quite well demonstrated. **Mr. Capp.**

The testing engineer of all professional men should face facts, yet it is characteristic of our profession that we are clinging to an attempt to continue a term which has only a very limited applicability and to make it apply generally in our testing practice.

Originally testing was done only on wrought iron and the milder sorts of structural steel. It is characteristic of these two materials that there is in the stress-strain curve a very sudden break or jog at a stress at which the specimen stretches considerably without any increase in stress. There was good reason for calling this point the yield point. When a material has this characteristic behavior, very rapid testing practice is possible. One has merely to insert the specimen in the testing machine, pull it at such a rate that the load can be accurately balanced, and then without any special apparatus the yield point becomes unmistakable. Furthermore, experience has shown that the yield point so indicated is not so far from the elastic or proportional limit in the vast majority of cases that the use of the yield point instead of the other values will lead the designing engineer into serious difficulties. There is, therefore, warrant in the determination of the yield point of metals which develop the definitely characteristic jog in the stress-strain curve.

There is only one reason for making tests of metal, that is, to determine the qualities of the metal for the purpose of providing information on which the metal may be applied in design. Surely there is no useful purpose served by supplying information which really has no intrinsic meaning. What purpose is served by reporting a yield point of 218,000 lb. per sq. in. from the data plotted in Fig. 2 of the paper or a yield point of 6150 lb. per sq. in. from the data plotted in Fig. 3? Neither value gives the designing engineer

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[1] Engineer, Testing Laboratory, General Electric Co., Schenectady, N. Y.

Mr. Capp. any useful information. Both values have to be obtained from the plotting of the stress-strain curve in which the proportional or elastic limit is indicated with sufficient accuracy to supply the designing engineer with really useful information. If the test is worth making with sufficient accuracy to determine the elastic or proportional limit, why cumber the report with a meaningless value? If the test is not worth making with sufficient accuracy to determine duplicable values which have some real meaning, then why not determine only the ultimate strength, elongation, and reduction of area, and let it go at that?

Mr. Moore. MR. H. F. MOORE[1] (*presented in written form*).—Mr. Lessells in his paper has suggested an interesting method of determining two arbitrary values at which, in a tension test, evidence of inelastic action reaches definite magnitude. There might be long discussion of the relative advantages and disadvantages of his method as compared with that proposed by J. B. Johnson and embodied in the Tentative Methods of Tension Testing referred to as Method II for determining elastic limit.

However, the writer wishes rather to comment on the significance of the values we determine and which we designate as "elastic limit," "proportional limit," or "yield point." All these terms designate a practical limit of *elastic strength* and the differences between them are differences in test method rather than in significance as a physical property.

If we could start our nomenclature afresh the writer would advocate the use of one term only, preferably the term "elastic strength," with the frank recognition in specifications that different methods were required for determining elastic strength for different materials, and that the elastic strength was a stress at which inelastic action did not exceed a small arbitrary tolerance.

In spite of the very great difficulty of changing an established nomenclature the writer wonders whether we may not come eventually to some such practice.

Mr. Rawdon. MR. H. S. RAWDON.[2]—At the U. S. Bureau of Standards we have just completed a study[3] of the strain markings on mild steel, some of the results of which have a direct bearing on this subject of the yielding of iron and steel under tension.

Briefly, 18-in. tension bars of mild steel of approximately 0.20-per-cent carbon content (56,700 lb. per sq. in. tensile strength, 33 per cent elongation in 8 in.), the surface of the reduced section of which had been polished, were stressed in tension and the beginning and growth of the strain markings observed as the bar was loaded and recorded photographically. Extensometer measurements were made on the same bar so that the full story of the behavior of the bar in tension was known. Everyone, no doubt, is familiar with the strain markings produced in this manner under the name of Lüders, Hartmann or Piobert lines. From the historical standpoint, the last name would seem the one to be preferred. Sketches were made of the general appearance of the strain markings produced in this manner, the propagation of the strain effect throughout a bar as it was repeatedly stressed in tension up to the point necessary to cause yielding. These illustrations, together with the stress-strain curves of the specimens repeatedly stressed in the manner indicated, and hardness surveys of similarly stressed bars are given in the reference cited above. **Mr. Rawdon.**

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[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[2] Senior Metallurgist, U. S. Bureau of Standards, Washington, D. C.

[3] H. S. Rawdon, "Strain Markings in Mild Steel Under Tension," Bureau of Standards *Journal of Research* Vol. 1, No. 3, September, 1928, p. 467.

By extensometer measurements the fact was definitely established that the appearance of the surface strain markings definitely corresponds to a permanent elongation of the bar. A permanent set in an 8-in. gage length as small as 0.004 per cent was found to result in the formation of unmistakable surface strain markings which, when the bar was repeatedly loaded up to the "yield point" became more pronounced by the widening and branching of the strain lines or bands and by the formation of new ones, thus resulting in the "propagation" of the effect along the length of the bar toward the central portion. Not until the bar had been strained sufficiently to give a permanent set of 1.5 per cent in 8 in., however, did the entire length of the reduced section show evidence of having "yielded" throughout.

The assumption that the extension of mild steel under tension is uniformly distributed over the gage length of the test bar is quite general. That this is true for elastic extension is not to be questioned and also it is more or less true for plastic flow such as occurs relatively late in the course of a tension test prior to "necking down." The initial "yielding" of the metal under stress, on the other hand, is quite different. This phenomenon is decidedly local in its character. In the ordinary tension test bar under load, the first strain markings, and hence, the first "yielding," occur at the ends of the reduced section close to the fillet. This effect might be predicted since, from the results of optical stress analysis, we know that the stress in these portions of a loaded tension bar is somewhat higher than it is elsewhere throughout the reduced section, where the average stress is very uniform across the bar. As the load is maintained,

Mr. Rawdon. perhaps at a slightly higher value than that at which the first lines appear, the effect at each end progresses along the length of the bar toward the center. It is very helpful to think of this effect somewhat as a "deformation wave" slowly proceeding along the bar. The rate at which the "wave" travels along the bar can be controlled by the rate at which the load on the bar is applied. It is a simple matter to obtain a bar which has "yielded" locally, that is, along only a portion of its length, by releasing the load on the specimen at the proper time. The main point which I wish to emphasize in this discussion, is that "yielding" of mild steel under tension is a phenomenon progressive in its nature. If a steel bar is strained sufficiently to produce a few strain markings near the fillet of the bar, it cannot be denied that the metal *in those portions of the specimen* reached the yield point just as truly as if the load had been maintained long enough so that the entire reduced section was covered with similar strain markings. Whether such a *bar*, however, would be said to have reached its yield point, depends entirely upon the definition to be applied to such a case.

The yielding of iron or mild steel under tensile stress, as is well known, results in the hardening and strengthening of the metal. This fact forms the basis of another method for demonstrating that "yielding" of mild steel is local in its character and that the effect progresses at a measurable rate (according to the conditions of loading) throughout the stressed bar. If the hardness of such a bar is determined along its entire length in its unstrained condition and the hardness redetermined after the bar has been strained sufficiently so as to cause some "yielding," it will be found that the hardness of the area in which "yielding" occurred, as shown by the strain markings, is considerably harder than before. The hardness of those portions of the strained bar, however, which show no strain markings will not be affected to any extent. The progress of the "yielding" effect along the bar can be followed in this way by determining the hardness after the bar as it is repeatedly strained to different amounts. It is worthy of note, that after yielding occurs (accompanied, of course, by an increase in the hardness) no further change in hardness of these yielded portions occurs when the load on the bar is again applied. Only in the new portions in which "yielding" occurs (as shown by the formation of new strain markings) will an increase in hardness be observed. After the "hardening wave" has progressed entirely along the bar, however, so that the entire reduced section has been affected, further application of load will result in plastic flow along the entire reduced section, as a result of which the entire length is hardened and strengthened.

The point that I want to illustrate and emphasize is that yielding of mild steel (in the form of the ordinary tension bar) under load is not general over the entire reduced section but is local and only gradually does the effect spread throughout the reduced section. Bearing in mind the real nature of the "yielding" of mild steel under tension, especially the fact that the effect is one which gradually moves along the length of the bar, it seems to the speaker that a rational definition of yield point for such a specimen is not possible, at least not practicable; any definition of yield point must be arbitrary in its nature. **Mr. Rawdon.**

MR. J. M. LESSELLS.[1]—Replying to Mr. Capp, I desire to point out that in determining yield point I was referring to Method II of the Society's Tentative Methods of Tension Testing of Metallic Materials (E 8 – 27 T).[2] I cannot agree with him that the yield point should be discarded and I think I can illustrate the importance of the yield point by the work that is being done in Germany on silicon steels. In these steels, with the introduction of silicon the ratio of yield point to tensile strength is considerably increased and in some cases may reach the value of 0.8 to 0.85. It would appear that for such a steel, under certain conditions, a higher working stress could be employed than for an ordinary steel of the same tensile strength. **Mr. Lessells.**

With regard to Mr. Moore's remarks, I think we are both on the same footing because he would suggest one definition. We are proposing that all of the others be discarded and the 0.2-per-cent plastic strain be adopted as the yield point. We would even discard, if necessary, the proportional limit values for normal temperature work.

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[1] Engineer in Charge, Mechanics Section, Research Laboratory, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 1074 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 732.

VOLUME CHANGES OF PORTLAND CEMENT AS AFFECTED BY CHEMICAL COMPOSITION AND AGING

By Alfred H. White[1]

Synopsis

This paper summarizes the results of tests on volume changes of cement which have been conducted over a period of 26 years. It shows graphically the volume changes of 12 bars of neat cement representing 6 brands of commercial cement and 34 expansion bars prepared from 16 cements burned in the laboratory covering a very wide range in composition.

Cements relatively low in alumina and high in iron oxide show the lowest volume changes with variations in moisture, but all cements undergo volume changes of material magnitude. Attention is called to the possible danger of liberating free magnesia in the burning process if the percentage of lime is raised too high.

Seventeen years ago it was the privilege of the author to present before this Society[2] evidence that structures made from portland cement not only expanded when kept in water, and contracted on exposure to air, but changed their length in unexpected measure when they were transferred from water to air or *vice versa*. Three years later, in a second paper before this Society,[3] it was shown that portland-cement structures retain their colloidal property of expanding when wetted and contracting when dried even after 20 years' exposure to the weather and after baking to 600° F.

The present paper discusses the effect of chemical composition of the cement on volume changes of bars made from neat cement. The data are drawn from measurements of volume changes of 11 commercial cements of ordinary composition and 2 commercial cements of special composition. A rather extensive study of the effect of aging was made on some of the cements. There are 68 expansion bars made from commercial cements whose history has been reviewed for this study, with ages ranging from 4 to 26 years. The effect of unusual compositions is shown by a study of 20 different mixes burned in our laboratory rotary cement kiln under carefully controlled conditions.

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[1] Professor of Chemical Engineering, University of Michigan, Ann Arbor, Mich.

[2] A. H. White, "Destruction of Cement Mortars and Concrete Through Expansion and Contraction," *Proceedings*, Am. Soc. Testing Mats., Vol. XI, p. 531 (1911).

[3] A. H. White, "Volume Changes in Portland Cement and Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XIV, Part II, p. 203 (1914).

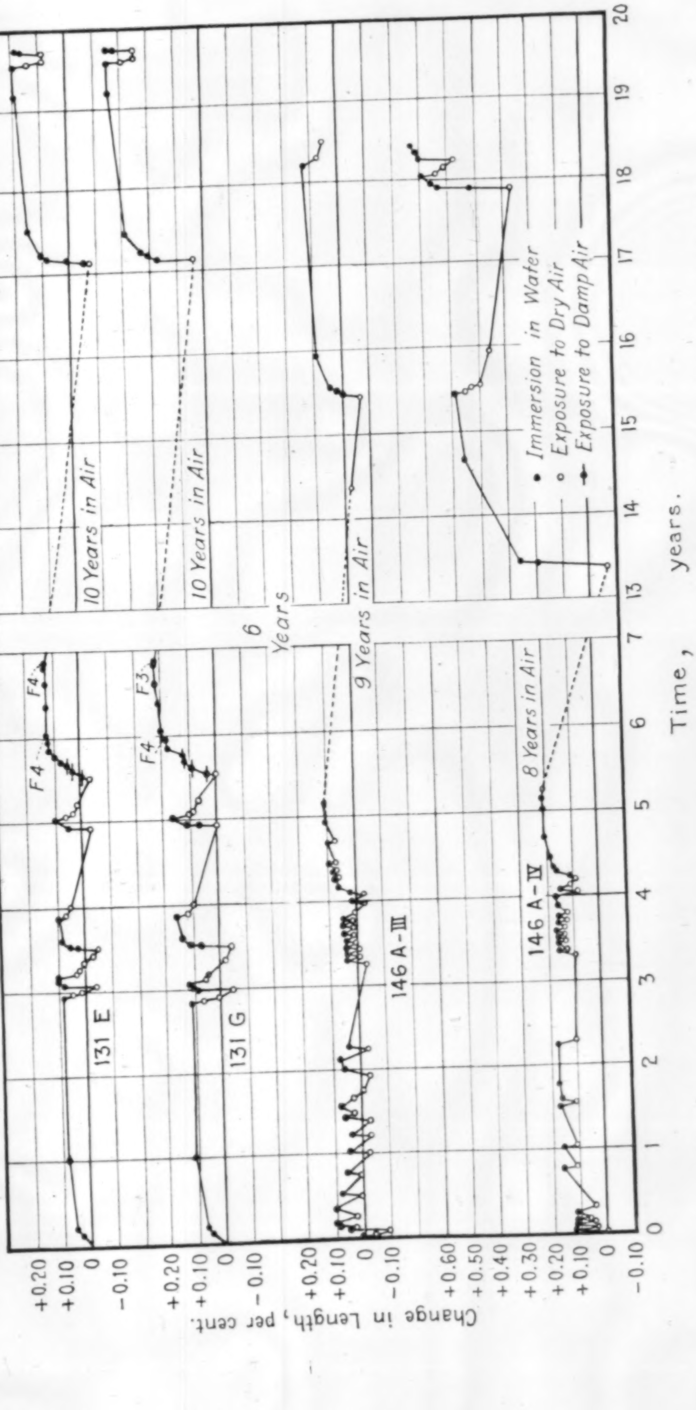

FIG. 1.—Volume Changes of Bars of Commercial Neat Cements with Change in Moisture.

Several of the mixes were burned in portions at different temperatures and some were aged. There are 85 expansion bars made from these laboratory cements which have been used in this study. Their histories cover a period of from 15 to 27 years. The properties of some of these cements were discussed 22 years ago in a paper[1] which gave details of burning, chemical composition, time of initial and final set and the manner of making and measuring the expansion bars. The slow expansion of the cements containing free magnesia after lying in water for periods of 10 to 14 years has also been presented previously.[2] Other papers by the present author[3, 4, 5, 6] have given further reports on the behavior of commercial portland cements with changes of moisture and on freezing, together with some theoretical explanation of the phenomena observed.

The chief value of the present paper is believed to lie in the variety of the materials studied and the long time over which the observations have extended.

VOLUME CHANGES IN COMMERCIAL CEMENTS OF USUAL COMPOSITION

The paper presented before this Society in 1914 showed curves of volume changes of a number of commercial cements all of which passed standard specifications. The history of four expansion bars presented in this earlier paper are completed to date in Fig. 1 of the present paper. Bar 131 *E* was made of the product from a small mill using marl and clay in the wet process, bar 131 *G* from a large mill using slag and limestone in the dry process, and 146 *A-III* and 146 *A-IV* from a large mill using the wet process. These bars all show the alternations in length with change in moisture content which are now recognized as typical of portland cements, with a progressive expansion following successive alternations between air and water.

Bars 131 *E* and 131 *G* have been given the same treatment throughout the 20 years of their life and the curves are very similar. However, from a quantitative standpoint the volume changes of bar 131 *G* are consistently greater than those of bar 131 *E*. After the first month in water the expansion of bar 131 *G* was 0.040 per cent while that of bar 131 *E* was 0.025 per cent. The effect of conditions during the mixing and during the first 24 hours of hardening might have caused differences of sufficient magnitude to invalidate any conclusions from these early measurements, but the divergencies between these two bars increased with time. During the second month bar 131 *G* grew 0.022 per cent while bar 131 *E* grew 0.018 per cent. During the next 10 months, bar 131 *G* grew 0.038 per cent while bar 131 *E* grew 0.028 per cent. After almost 3 years in water both bars were placed in air for 2 months. Bar 131 *G* shrank from +0.100 to −0.050 per cent, a total change of 0.150 per cent, while bar 131 *E* shrank from +0.073 to −0.060 per cent, a total of 0.133 per cent. At the end of 7 years bar 131 *G* had expanded to +0.241 per cent and in its nineteenth year to +0.339 per cent, while bar 131 *E* at corresponding periods had expanded to +0.124 and +0.184 per cent. The expansion of bar 131 *G* during these later periods is substantially double that of bar 131 *E* on the long swings. On the 60-day alternations shown at the end of the nineteenth year, the variations in length with each change in condition were substantially the same for both bars. This may indicate that the differences are lessening after a long time. It has been possible to compare these two bars *E* and *G* because they have been kept under identical conditions for 20 years.

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[1] Campbell and White, *Journal*, Am. Chemical Soc., Vol. 28, p. 1274 (1906).

[2] Campbell, *Industrial and Engineering Chemistry*, Vol. 8, p. 1101 (1916).

[3] "Volume Changes in Concrete," *Transactions*, International Engineering Congress, San Francisco, (1915), Volume on Materials of Engineering Construction, pp. 242-273.

[4] "The Fundamental Cause of the Disintegration of Concrete," *Proceedings*, Eleventh Annual Conference on Highway Engineering, at the University of Michigan, February, 1925. Reprinted in *Concrete*, May, 1925.

[5] "Hydrated Portland Cement as a Colloid," Fifth Colloid Symposium Monograph, p. 349-360 (1927).

[6] "Crazing in Concrete and the Growth of Hair Cracks into Structural Cracks," *Proceedings*, Am. Concrete Inst., Vol. 24 (1928).

The variations which may be caused by changes in conditions of storage are illustrated in the two lower curves of Fig. 1, which show the life history of two bars of another commercial cement. Bars 146 *A-III* and 146 *A-IV* were made at the same time, but subjected to different conditions after removal from the mold. Bar *A-III* was kept dry for the first 3 weeks and its length decreased to −0.071 per cent in that period. During the next 4 years it was alternated between the wet and dry states but was kept dry for relatively long periods, and, as shown in Fig. 1, at the end of 4 years its length was not far different from what its length had been when removed from the mold. Bar *A-IV* was immersed in water after removal from the mold and alternated on a schedule which gave relatively long periods in water. After 4 years its length was about +0.15 per cent while bar *A-III* was about zero. A subsequent attempt to make bar *A-III* grow by giving it a long immersion in water increased its length but, as Fig. 1 shows, it has not nearly caught up with bar *A-IV*, which shows the unusual expansion to a length of +0.647 per cent. The large expansion of bar *A-IV* during its thirteenth year may be due to rather abnormal conditions, since when it was first put into water in that year the penetration of water was accelerated by a vacuum. The theoretical explanation for these large and progressive changes in volume has been discussed in earlier papers by the author and will not be repeated here. So also the effect of dilution by aggregate will not be considered. This paper studies only the properties of the cements themselves.

THE EFFECT OF AGING SOUND CEMENTS

It is well known that the time of setting, and to some extent the soundness of portland cement, may be affected by aging the cement either as clinker or after grinding. It was thought that aging might also influence the volume changes and several series of tests were made to determine this.

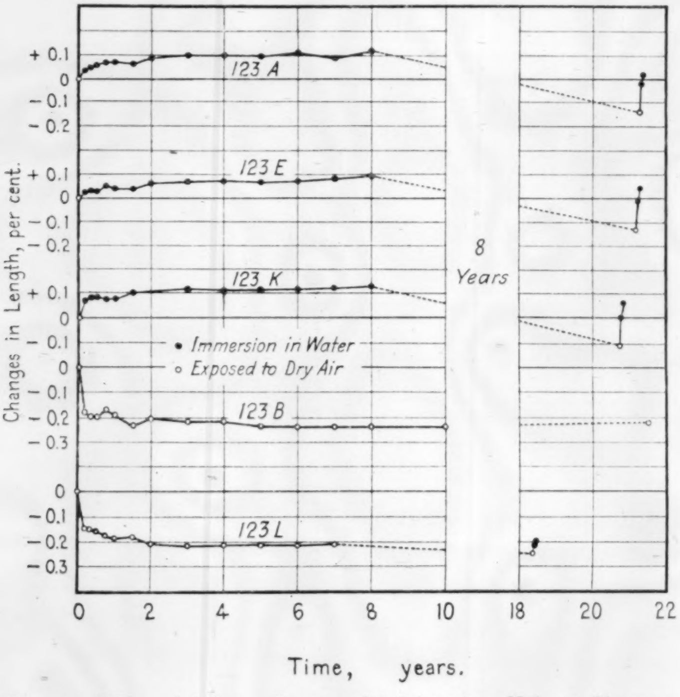

FIG. 2.—Effect of Aging a Commercial Cement on Volume Changes.

Bars *A* and *B* were made from fresh cement. Bar *E* was made after 1 month, bar *K* after 6 months, and bar *L* after 2 years and 6 months aging in a cloth sack in the laboratory.

There are four different brands of commercial cements on which aging tests were made. All give concordant results, but series 123 is selected because of its completeness. This cement was ground at the mill 4 days after it was burned and passed standard specifications at that time. The first expansion bar was made from neat cement on the day after it was ground and 10 other expansion bars were made after various periods of aging. The quantity of cement aged was about 50 lb. and it was kept in a duck sack at room temperature during the aging period. The results of 5 expansion bars of neat cement are shown in Fig. 2. Bars 123 *A* and 123 *B* were made one day after the clinker was ground, 22 per cent water being required to give normal consistency. Bar *E* was made after the cement had aged 1 month, bar *K* after 6 months and bar *L* after 2 years and 6 months aging of the cement in a cloth sack.

The aging has not made any noticeable difference in the volume changes. The three bars 123 *A*, 123 *E* and 123 *K* kept continually in water for 8 years showed the usual slow expansion. Bar *K*, which was aged 6 months, expanded a little more rapidly than the others, but the factors influencing the initial measurement are large enough to account for some fluctuations in the early history of the bars. The increase in length of the three bars after 8 years in water was 0.116, 0.093 and 0.125 per cent. After the bars had been stored in dry air for 13 years the decrease in length was 0.256, 0.223 and 0.237 per cent, respectively. On immersion in water for 60 days they expanded substantially equal amounts. Bar *B* made up 1 day after the clinker was ground and bar *L* made up 2 years and 6 months after the clinker was ground, were kept in air and showed substantially equal shrinkage.

The experiments on the other commercial cements and on those made in the laboratory all agree with this experiment and show that aging does not minimize the volume changes of cement which is initially sound.

CEMENTS OF UNUSUAL COMPOSITION

The effect of unusual chemical composition on volume changes is illustrated by two commercial cements, one high in iron and one high in alumina, and a number of cements made in the laboratory to determine the effect of variations in composition. Systematic experiments were made to determine the effect of high and low lime, free and combined lime and magnesia, high and low alumina, and high and low iron oxide.

Normal Cements Burned in the Laboratory:

Before presenting the results of abnormal compositions burned in the laboratory rotary kiln, it is advisable to show that cements of normal behavior were produced if normal mixes were used. Three examples are given, one of a mix rather high in lime, one low in lime and one of average composition. The first two were burned in the laboratory rotary kiln and the third in a fixed kiln which kept the clinker at the full furnace heat for 5 hours. Cement No. 112 was made from pure materials, calcium carbonate, glass sand and kaolin. The clinker had the following composition:

COMPOSITION OF CEMENT NO. 112.

	PER CENT	MOLS
SiO_2	22.60	100.0
Al_2O_3	11.15	29.4
Fe_2O_3	0.35	0.6
CaO	65.90	315.0

It will be noted that this cement is free from magnesia, contains very little iron but considerable alumina, and is high in lime. However, when burned at a temperature of 1520 to 1560° C. in the laboratory kiln and ground with 2 per cent of gypsum it passed the soundness test entirely satisfactorily, even when the cement was tested immediately after grinding. The volume changes of this cement were normal, as is shown by the graph 112 *C-5* of Fig. 3 where the behavior of a bar made from fresh cement is shown. Three other bars whose history is omitted because of lack of space confirm this behavior.

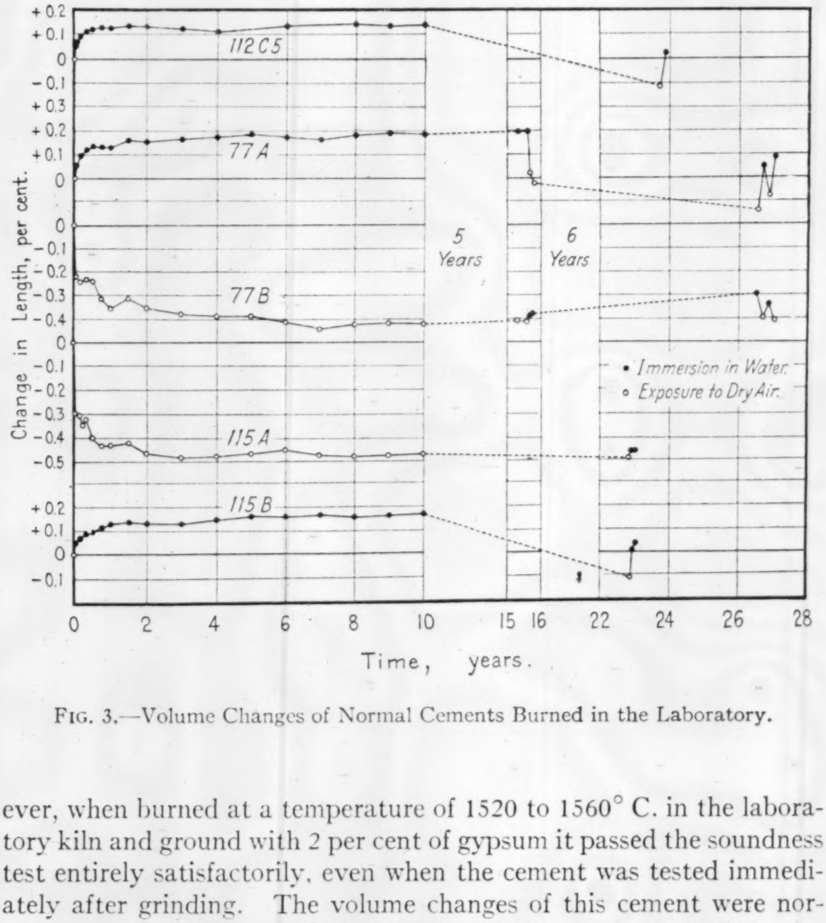

FIG. 3.—Volume Changes of Normal Cements Burned in the Laboratory.

Another cement designed to be a satisfactory cement rather low in lime carries our experiment number 77, the composition being as follows:

COMPOSITION OF CEMENT NO. 77.

	PER CENT	MOLS
SiO_2	24.52	100.
Al_2O_3	8.13	19.6
Fe_2O_3	2.79	4.3
CaO	61.66	270.7
MgO	2.89	17.7

This cement was made in the laboratory from marl and clay and was burned in the rotary kiln at 1522° C., which was as high as the temperature could be carried without causing the clinker to become sticky. It yielded sound cement even when ground fresh and the satisfactory history of bars 77 *A* and 77 *B* is shown in Fig. 3. Three other expansion bars made on aged cement showed similar history.

Series 115 was made to determine the effect of a longer burning period on the properties of the cement. The time required for the cement to traverse the laboratory rotary kiln varied with the pitch of the kiln and the rate of rotation, but was usually about one hour and a quarter. The cement was at the highest temperature, however, for only about 15 minutes. This cement was made from a mix sent us already ground by one of the cement mills and which had previously yielded satisfactory cement in our laboratory rotary kiln. It was burned as series 115 in a gas-fired fixed kiln where it was held for 5 hours at 1550 to 1580° C. and gave a sound cement which needed no gypsum to retard the set. The chemical composition follows:

COMPOSITION OF CEMENT NO. 115.

	PER CENT	MOLS
SiO_2	21.55	100.
Al_2O_3	9.09	24.9
Fe_2O_3	2.56	4.5
CaO	63.01	315.2
MgO	2.73	19.1

The graphs in Fig. 3 show the history of bars 115 *A* and 115 *B* made from fresh cement without the addition of any gypsum. They show normal expansion and contraction and the longer time of burning has not caused any apparent changes in the properties of the cement.

Cements Initially Unsound Because of Free Lime:

The manufacture of cement No. 77—a sound cement—was just described. In order to test the behavior of free lime in cement, a mix was prepared identical with No. 77 except that there was added to it 5 per cent of calcite crushed to pass a No. 12 but be retained on a No. 20 screen. It was certain that this coarse calcium carbonate would mostly appear in the clinker as lumps of uncombined lime, so that the fresh cement could be considered to contain about 2.7 per cent of free lime. The cement, No. 78, was unsound as was expected, but two expansion bars were made from the fresh cement, bar 78 *A* to be kept in water and bar 78 *B* to be kept in air. The behavior of these bars is shown in Fig. 4, but the small scale does not permit details to be seen. Bar 78 *A* expanded 0.220 per cent after only a week in water, while bar 77 *A*, shown in Fig. 3 and which was its duplicate except for the absence of free lime, expanded only 0.038 per cent in a similar period. After one month in water bar 78 *A* had expanded 0.295 per cent and bar 77 *A* only 0.070 per cent. Most of the free lime in bar 78 *A* had been hydrated after a month in water so that the curves run nearly parallel after that period, bar 78 *A* being on a

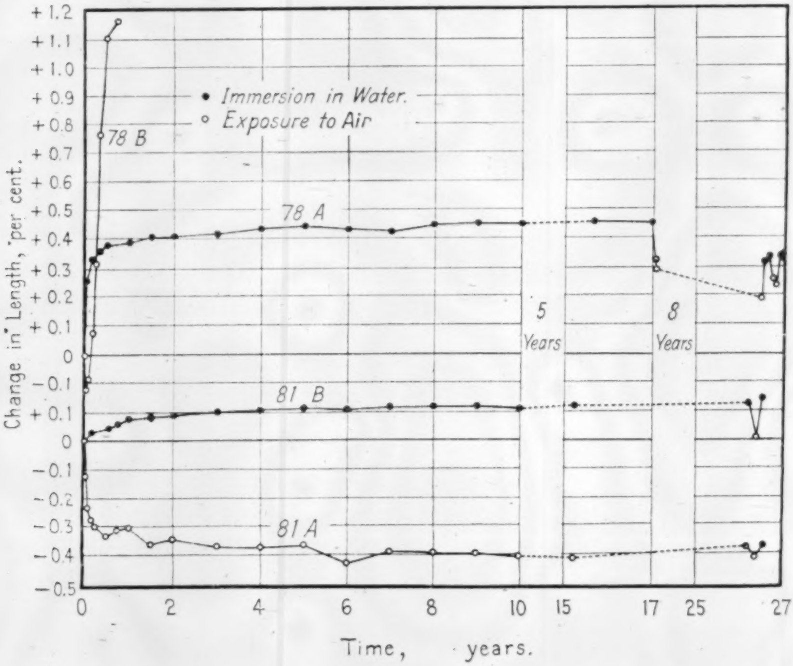

FIG. 4.—Effect of Aging in Elimination of Free Lime in Cements.

Bars 78 *A* and *B* show the behavior of a cement containing free lime.

Bars 81 *A* and *B* show the behavior of a cement, containing free lime when burned, which had the free lime hydrated by weathering the clinker so as to give a cement with normal volume changes.

higher level. After 16 years in water bar 78 *A* and 77 *A* were both transferred to air for 11 years. During this long period in air and on subsequent alternations between air and water they behaved similarly. The effect of the free lime was apparent during the first 2 months, and hydration was substantially complete at the end of that period.

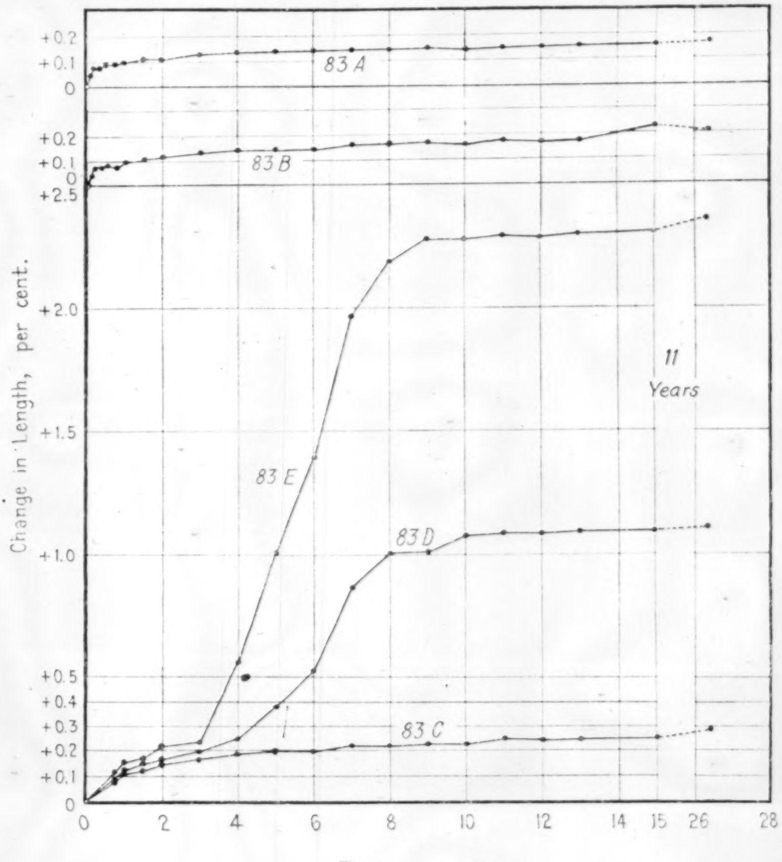

FIG. 5.—Effect of Free Magnesia on Expansion of Neat Cement in Water.

Bar *A* shows the behavior of a normal cement. The other bars show the expansion of this same cement after addition of varying amounts of hard-burned magnesia. Bars *B*, *C*, *D* and *E* had added to them 1, 2, 3, and 4 per cent magnesia, respectively.

Bar 78 *B* containing free lime was kept in the air of the laboratory. It was made in March when the humidity was commencing to rise with warmer weather. It contracted normally for the first week, but then started to expand as the lime hydrated. After 28 days in air its decrease in length was still 0.085 per cent, as shown in Fig. 4, but after 2 months its length was greater than it had been initially. The expansion increased progressively until after 9 months in the air of the laboratory its increase in length was +1.170 per cent and measurements had to be discontinued because the bar was badly cracked. The interesting thing about this bar was the evidence that cements containing free lime expanded even in the air and that the slow hydration of lime in the air caused more marked expansion than the hydration of lime in water.

In order to determine how long cement had to be aged in air in order to hydrate the free lime and give a sound cement, cement No. 81 was burned as a duplicate of No. 78, carrying 2.7 per cent of free lime. The clinker was exposed to the weather until, after four months of exposure in an open box on the roof in the summer weather, tests of small portions showed the cement to be sound. The clinker was then ground with 2 per cent of gypsum and bars 81 *A* and *B* were made, *A* to be kept in water and *B* in air. The life history of these bars is shown in Fig. 4. The aging has eliminated the free lime and left an entirely normal cement. Seven other expansion bars were made from this same cement to test the effect of further aging, but there is nothing noteworthy in their history.

Free and Combined Magnesia:

In some of the earlier papers to which reference has already been made, it was shown that magnesium oxide burned at the temperature of the portland cement kiln hydrated with extreme slowness even when finely ground. Tests made by mixing known amounts of finely ground, hard-burned magnesium oxide with a normal commercial portland cement gave a quantitative measure of its effect on expansion. These tests of series 83 are shown in Fig. 5. The original cement bar 83 *A* showed a normal expansion in water; bar *B* with 1 per cent added magnesia showed only slightly greater expansion; bar *C* with 2 per cent added magnesia showed only a moderate effect even after 26 years in water. Bar *D* with 3 per cent added magnesia was almost normal at the end of the first year and had not shown large expansion even after 3 years in water. There was marked expansion for the next 5 years but the hydration was apparently substantially complete after 8 years, for the growth was slight during the next 18 years so that its present increase in length, +1.099 per cent, is substantially what it showed 10 years ago.

Bar 83 *E* with 4 per cent added magnesia did not show marked expansion until after 3 years in water. It then started to expand rapidly and at the end of 8 years had reached an expansion of +2.185 per cent. Now, after 26 years in water, it has an expansion of +2.360 per cent. The normal expansion of the bars with 1 and 2 per cent of magnesia may be due to voids in the bars, even of neat cement, which

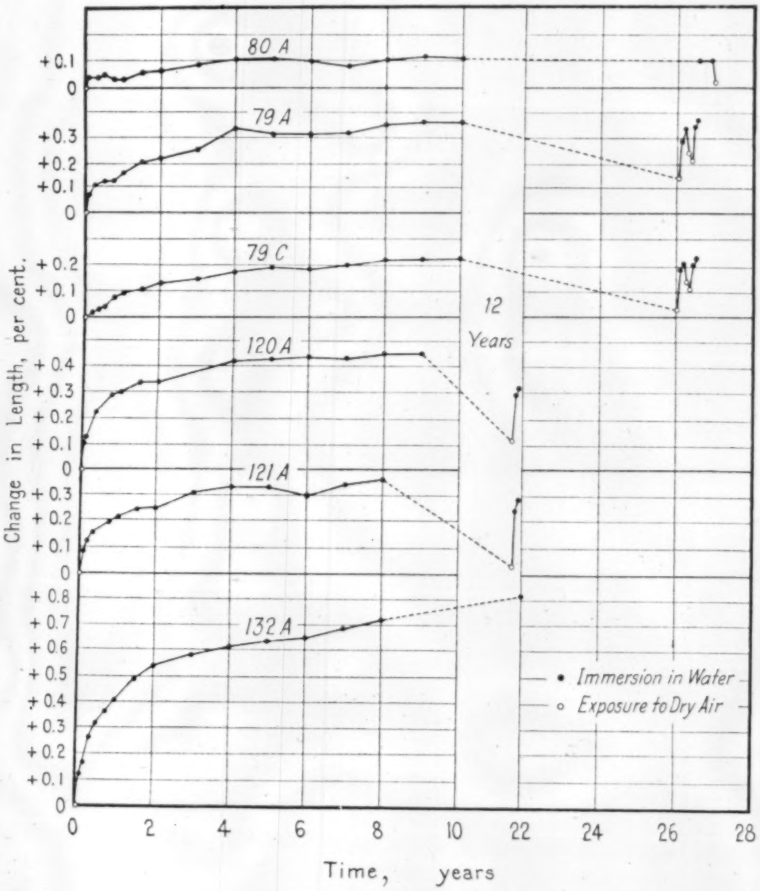

FIG. 6.—Effect of Free and Combined Magnesia on Volume Changes of Portland Cement.

Cement No. 80 contains 53.07 per cent MgO, but all in combination.

Cement No. 79 contains 7.21 per cent MgO, with a small amount of free lime and a small amount of free magnesia. Bar *A* was made from fresh cement and bar *C* from aged cement, from which the free lime had been eliminated.

Cements Nos. 120 and 121 contain 4.45 per cent MgO. Both contain small amounts of free lime and magnesia.

Cement No. 132 contains 8.25 per cent MgO with considerable magnesia in the free state.

permit a certain amount of internal expansion. After these voids have been filled, the additional increase in volume due to hydration of more magnesia must be directed externally.

The conclusions from these curves are that free magnesia cannot be detected with certainty by expansion measurements on bars of neat cement lying in water until after the third year, although a steady increase in length after the first year is suspicious. With large percentages of free magnesia the main expansion occurs between the third and the tenth years, but a slow expansion seems to be continuing even after 26 years continuous immersion in water.

These experiments had given a measure of the quantitative effects of free magnesia on expansion in water. The next step was to try and determine what proportion of the magnesium oxide in normal cements existed in the free form and what proportion was in combination with silica or alumina and therefore probably harmless. The most striking instance of high combined magnesia is found in cement No. 80, which was made from clay and pure magnesium carbonate, the only lime in the cement coming from the clay. The chemical composition of the clinker was as follows:

COMPOSITION OF CEMENT NO. 80.

	PER CENT	MOLS
SiO_2	28.91	100.
Al_2O_3	9.79	20.
Fe_2O_3	3.14	4.1
CaO	5.08	18.9
MgO	53.07	274.7

This was burned in the laboratory rotary kiln at 1496° C. and yielded a hard clinker which, when ground with 1.5 per cent of gypsum, required 23 per cent of water for normal consistency and took its initial set in 52 minutes and its final set in 9 hours. The pat from the soundness test was perfect but weak. One expansion bar, 80 *A*, was made from this neat cement and its history is shown in Fig. 6. It will be noted that in spite of its 53 per cent of magnesia it showed a rather low normal expansion even after 26 years in water. Evidently all of its magnesia was in harmless combination. Briquets made from this cement showed a tensile strength of 43 lb. per sq. in. at 28 days, 114 lb. at 1 year and 162 lb. at 2 years. This cement is not to be recommended as a commercial material but it shows the possibility of forcing even large amounts of magnesia into some form of combination where it is harmless. As a check on this result another cement was made up from the same formula and burned, yielding a cement which also showed a low normal expansion.

After it had been thus shown that high percentages of magnesia could be made to enter into combination and give a cement with low expansion, provided the lime was kept sufficiently low, it became a question of determining how much lime and magnesia could be combined without causing either free lime or free magnesia. In the earlier pages of this paper cement No. 77 has been discussed as a sound cement somewhat low in lime. Cement No. 79 was made from the same formula as cement No. 77 except that enough magnesium carbonate was added to raise the MgO from 2.89 to 7.21 per cent. Its composition follows:

COMPOSITION OF CEMENT NO. 79.

	PER CENT	MOLS
SiO_2	23.43	100.
Al_2O_3	7.77	19.6
Fe_2O_3	2.67	4.3
CaO	58.92	270.7
MgO	7.21	46.

It passed the soundness test but required 2.5 per cent of gypsum to retard the set. The history of bar 79 *A* which was made from this fresh cement is shown in Fig. 6. The rather high expansion during the first month indicates a little free lime and the steady expansion between the first and fourth years shows free magnesia. A portion of the clinker of cement No. 79 was stored on the roof exposed to the weather for 6 months and was then ground to produce bar 79 *C*, whose history is also given in Fig. 6. This aging had eliminated the free lime and the cement showed only the slow and progressive expansion due to the free magnesia, which must have been small for the total expansion of this bar after 16 years in water was only +0.261 per cent.

The previous experiments on synthetic cements containing magnesia had mainly used magnesium carbonate as the source of the magnesia. It was now desired to determine how far free magnesia would be liberated from a silicate during burning, and cements Nos. 120 and 121 were made to test this displacement. A cement was made from talc, kaolin and marl to be rather high in lime. The composition of the clinker is shown below:

COMPOSITION OF CEMENTS NO. 120 AND NO. 121.

	PER CENT	MOLS
SiO_2	20.80	100.
Al_2O_3	10.29	29.1
Fe_2O_3	0.51	0.9
CaO	63.95	330.
MgO	4.45	30.8

Cement No. 120 was burned in the rotary kiln extremely slowly in an effort to attain equilibrium at a kiln temperature of 1500 to 1550° C. An expansion bar 120 *A* made from this product and presented in Fig. 6 showed an expansion in water of +0.089 per cent at the end of 7 days and +0.301 per cent at the end of a year, indicating free lime. The continued expansion after the first year also indicated free magnesia. A second burning was made of this same mix at a customary rate of passage through the rotary kiln and bar 121 *A* made from this fresh cement showed an expansion after 7 days in water of only +0.015 per cent, indicating the absence of free lime. The history of both these expansion bars is given in Fig. 6. It will be observed that both bars show the presence of free magnesia, but only cement No. 120, which was burned more slowly, shows free lime. The significant thing about this test is that though the cement contains only 4.45 per cent total magnesia and all of it was present as a silicate in the raw material, the rather high lime has caused sufficient magnesia to be set free to cause abnormal expansion. It indicates that caution should be exercised in increasing the lime content of commercial cements for fear that free magnesia may result.

Another cement, No. 132, was made from the same raw materials, but with somewhat higher magnesia and lower lime. The analysis follows:

COMPOSITION OF CEMENT NO. 132.

	PER CENT	MOLS
SiO_2	23.85	100.
Al_2O_3	6.78	17.
Fe_2O_3	1.95	3.1
CaO	59.15	267.7
MgO	8.25	52.

This cement burned at 1550 to 1564° C. did not show free lime on the steaming test, but an expansion bar 132 *A* made from fresh cement showed an expansion of +0.052 per cent after 3 days and +0.125 per cent after 28 days in water, indicating the presence of some free lime. After 8 years in water the expansion was +0.725, showing the presence also of free magnesia. The curve is given in Fig. 6.

Another cement, No. 130, made from the same raw materials, kaolin, talc and pure calcium carbonate, but with higher lime, was made as part of this series. The composition is given below:

COMPOSITION OF CEMENT NO. 130.

	PER CENT	MOLS
SiO_2	23.21	100.
Al_2O_3	6.54	16.7
Fe_2O_3	0.56	0.9
CaO	62.25	286.7
MgO	7.60	48.4

This cement was burned at 1548 to 1551° C. but gave a very unsound cement, the pat for the soundness test disintegrating completely. An expansion bar was made which after 7 days in water expanded to +4.842 per cent and after a year to +5.745 per cent, indicating a large amount of free lime. A part of this same cement was aged in the air of the laboratory for 11 months to eliminate the free lime. An expansion bar made from this aged cement gave an expansion of 0.144 per cent after 7 days, and +1.161 per cent after a year in water, showing that it still carried free lime. This cement evidently carried both free lime and free magnesia. The curve is not shown on the graphs.

A study of these three cements made from talc, kaolin and pure calcium carbonate shows clearly that there is replacement of magnesia of the talc by lime. A tabulation of the molecular ratios of the six laboratory cements used in this study of magnesia gives an interesting insight into the correlation between chemical composition and expansion in water. This is given in Table I. It will be noted that free magnesia has appeared whenever the molecular ratios of basic to acidic constituents has risen above 2.5. This may not be the case in commercial practice and free magnesia does not exert nearly as strong an effect in concrete as it does in neat cement, because of the dilution by the aggregate and the greater quantity of voids available for internal expansion. It is believed, however, that care should be taken in increasing the lime in portland cements over that which experience of the past has shown to be safe.

TABLE I.—BEHAVIOR OF MgO IN CEMENTS.

Cement	Composition in Mols							Ratio: base to acid
	CaO	MgO	Al_2O_3	Fe_2O_3	SiO_2	Total R O	Total Acid	
No. 80	18.9	274.7	20.0	4.1	100	293.6	124.1	2.36
No. 79	270.7	46.0	19.6	4.3	100	316.7	123.9	2.55
No. 120	330.0	30.8	29.1	0.9	100	360.8	130.0	2.77
No. 121								
No. 132	267.7	52.0	17.0	3.1	100	319.7	120.1	2.66
No. 130	286.7	48.4	16.7	0.9	100	335.1	117.6	2.85

	Expansions				
	1 Year in Water	Long Time in Water	Increase on Long Time	Ratio: base to acid	Quality of Cement
No. 80	+0.032	+0.102 (16 yrs.)	0.070	2.36	O.K.
No. 79 A	+0.160	+0.377 (10 yrs.)	0.217	2.55	Free MgO
No. 120	+0.301	+0.459 (9 yrs.)	0.158	2.77	Free CaO and MgO
No. 121	+0.215	+0.365 (8 yrs.)	0.150		
No. 132	+0.414	+0.725 (8 yrs.)	0.311	2.66	Free CaO and MgO
No. 130 aged	+1.161	+1.447 (8 yrs.)	0.281	2.85	Free lime and some free MgO

EFFECT OF VARIATIONS IN IRON OXIDE

The effect of very low percentages of iron oxide may be noted in cement No. 112 which has already been mentioned. There is nothing to differentiate this cement from others of normal composition. Attempts to make cements which were essentially calcium ferrites yielded such weak materials that the investigation was not carried further. There are, however, two laboratory cements and one commercial cement which are high enough in iron to merit consideration.

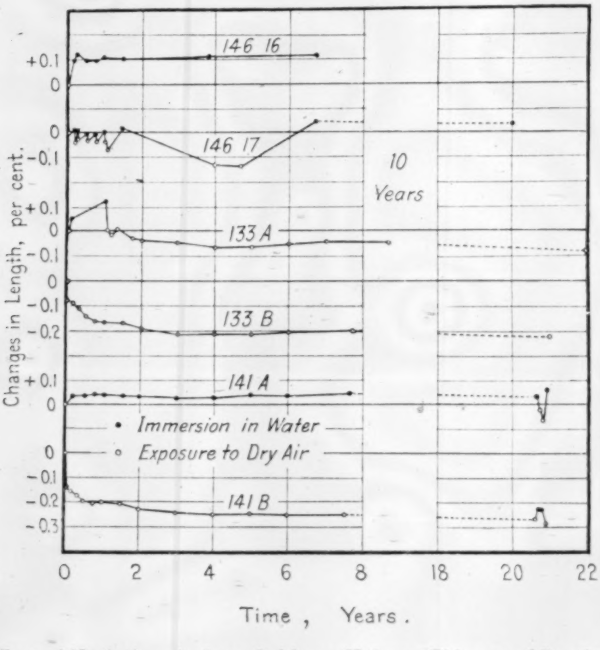

FIG. 7.—Effect of Variations in Iron Oxide on Volume Changes of Portland Cement.

Cement No. 146 is a commercial iron ore cement.
Cement No. 133 is a laboratory cement containing 6.90 per cent Fe_2O_3.
Cement No. 141 is a laboratory cement containing 6.23 per cent Fe_2O_3.

Series 146 was made to test the properties of a commercial cement advertised as an iron-ore cement. Only a partial analysis was made of it with the following results:

PARTIAL ANALYSIS OF CEMENT USED IN BARS 146-16 AND 146-17.

SiO_2	23.9	per cent
Al_2O_3	0.3	"
Fe_2O_3	11.8	"

Four bars of neat cement were made from this material and the history of two of them is shown in Fig. 7. Bar 146-16 expanded 0.107 per cent during its first year and 0.111 per cent during its first 7 years in water. Bar 17 alternated between a wet and dry schedule, but its variations were on the whole small. This cement has been more constant in volume than the average cement.

Cement No. 133 was burned in the laboratory from commercial materials rather high in iron oxide as shown by the following analysis:

ANALYSIS OF CEMENT NO. 133.

	PER CENT	MOLS
SiO_2	22.55	100.
Al_2O_3	5.88	15.4
Fe_2O_3	6.90	11.5
CaO	62.50	298.4
MgO	1.01	6.7

The cement was burned at 1500 to 1515° C. and was sound. Bar 133 *A* was made from fresh cement, but bar *B* was not made until the cement had aged a year. The volume changes of this cement are also rather small.

Cement No. 141 was synthesized to find the effect of replacing part of the alumina by iron oxide. Cement of the following composition was burned at 1540 to 1550° C. and gave a sound and very strong pat after steaming.

COMPOSITION OF CEMENT NO. 141.

	PER CENT	MOLS
SiO_2	23.30	100.
Al_2O_3	3.08	7.5
Fe_2O_3	6.23	10.1
CaO	64.40	297.5
MgO	2.99	19.2

The behavior of bars 141 *A* and 141 *B* made from fresh cement are shown in Fig. 7. The volume changes are unusually small. Four other bars made from this same cement after aging give the same picture.

All three of these cements indicate that cements rather high in iron oxide show small variations in length with changing moisture content.

PROPERTIES OF CALCIUM SILICATE

An attempt to synthesize a sound cement which should consist almost entirely of calcium silicate caused many failures. It was noted, however, that a small amount of alumina assisted the clinkering process and a sound cement, No. 144, was finally prepared from pure calcium carbonate and glass sand burned in the rotary kiln at about the maximum attainable temperature of 1578° C. This clinker did not dust in the air, required 45 per cent of water for normal consistency and took an initial set without added gypsum in 6 hours and a final set in 9 hours. The microscope showed a small amount of lime by the phenol test. The soundness test showed a pat which came loose from the glass but was not appreciably warped. The composition of this cement is given below:

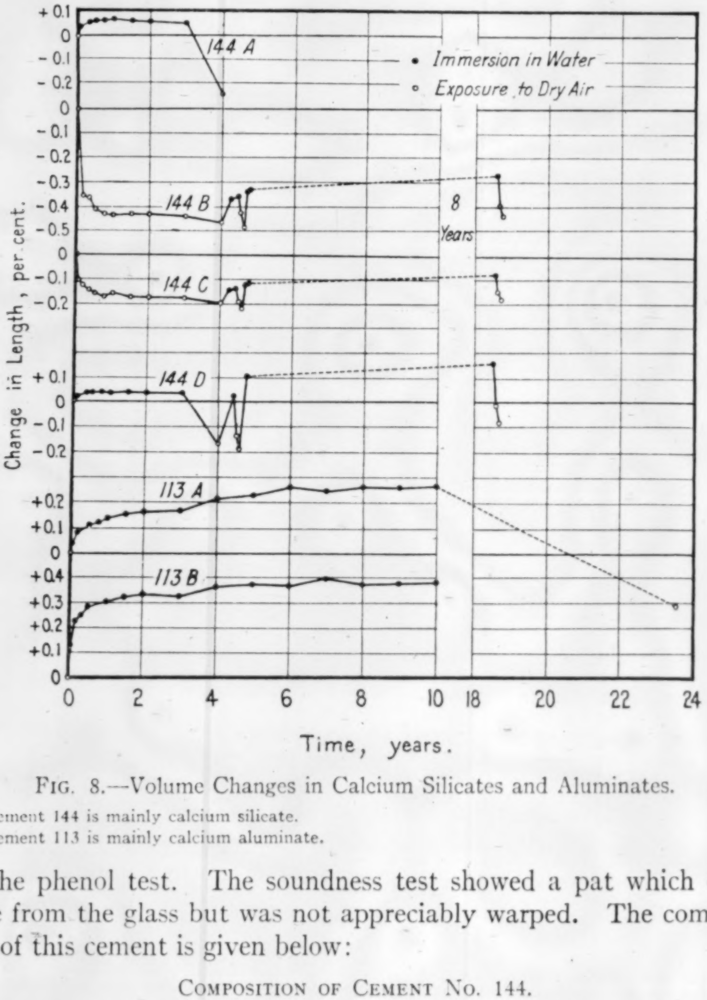

FIG. 8.—Volume Changes in Calcium Silicates and Aluminates.

Cement 144 is mainly calcium silicate.
Cement 113 is mainly calcium aluminate.

COMPOSITION OF CEMENT No. 144.

	PER CENT	MOLS
SiO_2	26.74	100.
Al_2O_3	2.77	6.1
Fe_2O_3	0.11	0.1
CaO	70.06	282.1

Bars 144 *A* and 144 *B* were made from this fresh cement. Their histories are given in Fig. 8. Bar *A* showed rather low expansion in water and on placing in air showed a normal shrinkage. The bar became broken after 4 years. Bar *B* placed initially in air shows a normal behavior.

In an effort to make the cement more homogeneous the ground clinker was reburned in the cement kiln. This twice-burned cement did not show any free lime in the microscopic test and took its initial and final set in about the same time as that from the first burning. Expansion bars *C* and *D* made from this twice-burned cement are shown in Fig. 8. Their volume changes are smaller than those of bars *A* and *B*. The expansion of these calcium silicates in water is low and their contraction in air is about normal. The volume changes with variations between water and air are about the same as with other cements of customary composition.

PROPERTIES OF CALCIUM ALUMINATES

A mixture was made from pure calcium carbonate and alumina to yield a clinker with the molecular formula $2\ CaO \cdot Al_2O_3$. The alumina carried some silica so that the composition of cements Nos. 111 and 113 was actually as follows:

COMPOSITION OF CEMENTS NO. 111 AND NO. 113.

	PER CENT	MOLS
SiO_2	2.1	3.3
Al_2O_3	45.5	44.6
CaO	52.3	93.4

Samples collected from the kiln while cement No. 111 was being burned did not show sound cement until the overburning temperature of 1382° C. was almost reached. Neither cement No. 111 nor No. 113 was entirely satisfactory in its behavior in setting. The original notes record that the bars seemed to form a hard crust on the outside while they were still soft on the inside. The history of two bars is given in Fig. 8. Bar 113 *A* was made from fresh clinker ground with 3 per cent of gypsum. It was kept in water for 10 years with slow continuous expansion and then in air for 13 years with rather high shrinkage. When it was put into water at the end of a long period in air it spalled so badly that further measurements could not be made. Bar 113 *B* was made from some of the same ground cement after it had been stored open in the laboratory for 10 months. For some unexplained reason this bar 113 *B* from aged cement showed greater volume changes than bar *A*, which was made from fresh cement. This bar also spalled when it was placed in water after a long period in air.

CEMENTS HIGH IN ALUMINA

One example of a normal cement rather high in alumina and low in iron oxide has already been considered in the discussion of cement No. 112. Cements Nos. 77 and 115 were also somewhat high in alumina. The curves show rather rapid expansion of these bars during the first 6 months in water, although the total volume changes were not more than normal. The aluminates are supposed to be the constituents responsible for the early strength of cement and this rather rapid expansion is quite possibly associated with the early gain in strength.

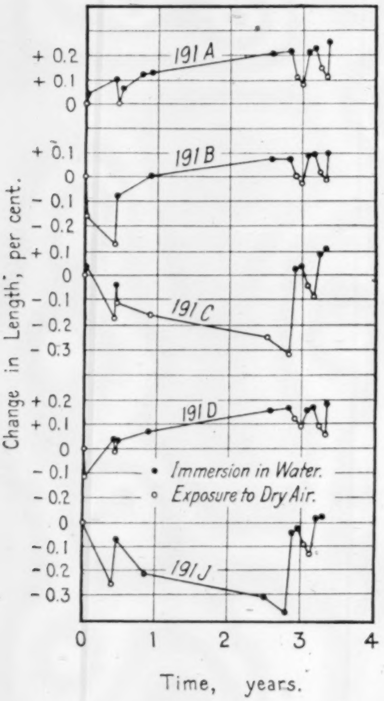

FIG. 9.—Volume Changes in a Commercial Cement High in Alumina.

The other example available for this study is a well-known commercial high-alumina cement which has been designated No. **191** and whose chemical analysis is given below:

COMPOSITION OF COMMERCIAL CEMENT NO. 191.

SiO_2	8.50	per cent
Al_2O_3	37.24	"
Fe_2O_3	18.34	"
CaO	31.60	"
MgO	1.13	"
SO_3	1.47	"
Loss on ignition	2.08	"

The behavior of this cement has been studied in seven expansion bars of neat cement and four bars of 1:3 mortar, all, however, made from one lot of cement. The history of five of these bars of neat cement is shown in Fig. 9. The volume changes are rather large. This is particularly the case when the bar is immersed in water after having been thoroughly dried, as is illustrated in bars 191 *C* and 191 *J*. The scale of the curves is too small to show the volume changes during small time intervals, but the data show that the two bars 191 *C* and 191 *J* when immersed in water after 2 years in air expanded 0.10 per cent on immersion in water for only one-half hour and 0.21 per cent after immersion for three hours, whereas the two commercial cements No. 131 *E* and No. 131 *G* of Fig. 1 expanded only 0.03 per cen ton immersion for three hours after having been in air for a year between the fourth and fifth years of their existence.

CONCLUSIONS

Expansion in water and contraction in air seem to be universal characteristics of portland cement of even the most diverse chemical compositions. This is shown by graphs of 12 expansion bars prepared from 6 neat commercial cements, and 34 expansion bars prepared from 16 cements burned in the laboratory, and the conclusions are reinforced by an equal quantity of material which has not been presented because of lack of space. Most of the bars have been under observation over 20 years.

The extent of the volume changes is influenced to a considerable extent by the conditions surrounding the material during the early weeks of the hardening process. Neat cement bars which lie in water continuously all show slow expansion for 2 years and this expansion may amount to +0.15 per cent. Good cements should show substantially no further expansion if they are kept in water for a longer time. Bars of neat cement which are allowed to dry in air immediately after removal from the molds all shrink, unless they contain free lime, and the shrinkage amounts to between 0.3 and 0.5 per cent.

If bars of neat cement which have been in water are transferred to air, they will all shrink from 0.2 to 0.3 per cent if sufficient time is given. The reaction is slow and bars only 1-in. square will not shrink over half the total possible amount in 60 days. Bars which have been placed in air immediately after removal from the molds and allowed to become thoroughly dry in air, expand rather slowly when placed in water. If their colloid has been previously developed by long immersion in water they will expand much more rapidly on being transferred from air to water. All these factors must be taken into account in determining the influence of chemical composition upon volume changes, but certain factors seem clear.

Free lime causes an unusual expansion during the first few weeks in water, but its hydration should be finished in a year and if the amount of free lime was small, the volume changes beyond that time should be normal. If bars of neat cement containing much free lime are placed in air after removal from the molds, there will be the usual contraction for a few weeks but then expansion will take place due to the hydration of the lime, and the expansion in air will be greater than the expansion would have been if the bar had been kept in water. Exposure of clinker to the weather, or of ground cement to indoor air, will hydrate the free lime in a few months and the cement which has been thus aged will show normal volume changes.

Magnesia in combination as silicate is entirely harmless, no matter how much there may be in the cement. Free magnesia hydrates slowly and causes rather slight additional expansion during the first 3 years in water; but if much magnesia is present, it causes marked expansion during the period from the third to the tenth year in water. Slow expansion persists for over 25 years. There is danger of free magnesia in cement even if all of the magnesia was present in the raw material as silicate, since lime may displace magnesia from combination during the burning process. The higher the lime, the greater is the tendency to liberate magnesia and there is evidence that a cement with only 4.45 per cent total magnesia may have enough in the free form to cause undue expansion. Magnesia does not give trouble through expansion if the cement products are kept continuously in air.

Increasing the iron oxide in a cement seems to decrease volume changes. The cements which have been most constant in volume are those which are rather low in alumina, and contain more iron oxide than alumina. Cements high in alumina have shown rather high volume changes. The one experimental cement which was essentially a calcium silicate showed small expansion when kept continually in water but shrank as much as a normal cement in the air.

Aging of a cement which contains free lime will remove the harmful effect of the free lime, but aging does not eliminate the normal volume changes of a cement nor does it seem to lessen them in any material degree.

The data presented in this paper suggests that care should be taken in increasing the percentage of lime in portland cements for fear of liberating free magnesia, and indicates that cements low in alumina and rather high in iron oxide are likely to have smaller volume changes than many of those which have been tested.

DISCUSSION

Mr. Bogue.

MR. R. H. BOGUE[1] (*presented in written form*).—The experiments reported by Mr. White have extended over a period of 27 years and were so designed at the beginning that many important types of information have been obtained in the study of them. For these reasons they are unique and extraordinarily valuable.

At the time when these studies were initiated very little was known about the compounds present and no method was available for the determination of free lime. In the past few years we have learned the nature of the compounds which can be present at equilibrium in a system approaching portland cement in compositions made from the components lime, alumina, silica, magnesia, and ferric oxide. A reconsideration of the data presented by Mr. White on the basis of the calculated compositions is both interesting and instructive.

TABLE I.—CALCULATED COMPOSITIONS OF A NUMBER OF CEMENTS REFERRED TO IN THE PAPER BY A. H. WHITE

	$3CaO \cdot Al_2O_3$, per cent	$4CaO \cdot Al_2O_3 \cdot Fe_2O_3$, per cent	$4CaO \cdot 2MgO \cdot Al_2O_3 \cdot Fe_2O_3$, per cent	$2CaO \cdot SiO_2$, per cent	$3CaO \cdot SiO_2$, per cent	CaO, per cent	MgO, per cent	$5CaO \cdot 3Al_2O_3$, per cent
No. 112	29.0	1.06	0	49.0	20.94	0	0	0
No. 77	5.6	0	9.8	73.0	0	0	1.5	10.1
No. 115	19.9	0	8.8	40.9	28.0	0	1.4	0
No. 79	16.2	0	9.6	63.4	5.0	0	5.8	0
No. 120–1	26.5	0	1.8	36.0	31.6	0	4.2	0
No. 132	14.6	0	7.1	59.6	11.6	0	7.3	0
No. 130	16.7	0	2.0	43.6	30.6	0	7.3	0
No. 133	4.0	....	22.0	39.9	33.0	0	0	0
No. 141	0	....	17.9 (+2.2 C_2F)	26.7	53.2	0	0	0
No. 144	7.2	0.4	0	29.1	63.0	0	0	0
No. 111-13	17.0	0	0	6.0	0	0	0	76.9

In the accompanying Table I are shown the calculated compositions of a number of the cements referred to in Mr. White's paper. The compositions given by Mr. White as typical of a high-quality cement showing low expansion in water are Nos. 112, 77, and 115. It will be observed that these compositions contain no free lime and very small amounts of free magnesia.

Of the cements high in magnesia, with the exception of No. 80, the content of free magnesia at equilibrium varies from 4.2 to 7.3 per cent. All of these cements are shown by Mr. White to expand a few tenths of one per cent in the course of a number of years when immersed in water in accordance with the procedure outlined in the

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[1] Research Director, Portland Cement Fellowship, U. S. Bureau of Standards, Washington, D. C.

Mr. Bogue.

paper. With reference to cement No. 80, it contained 53 per cent of MgO and only 5 per cent of CaO, from which it may be pointed out that the actual compounds present are not at all the same as those which are present in a portland cement. Consequently any deductions drawn from that composition can have no bearing on the examination of the behavior of portland cements.

Mr. White is led to believe that the magnesia is forced out of combination by lime and that in a cement which may contain only a small amount of magnesia, that magnesia may be forced out of combination by increasing the lime. Our present knowledge of the systems concerned in portland cement indicates that the magnesia goes into compositon as the compound $4CaO \cdot 2MgO \cdot Al_2O_3 \cdot Fe_2O_3$. In the usual portland cement, the amount of that compound is limited by the amount of ferric oxide present. Magnesia beyond that which can go into this combination appears to remain as free MgO. This would indicate that any increase in lime, in compositions approaching that of portland cement, will have no effect on the amount of combined or uncombined magnesia. The ferric oxide present in the four cements examined for free magnesia was low in each case, hence, the products contained fairly large amounts of free MgO. If compositions were prepared containing the same amount of CaO and MgO but more Fe_2O_3, the free MgO in the products would be decreased and probably the expansion between 3 and 10 years would be less.

The experiments of Mr. White indicate that free lime in cements may result in some expansion in water during the first few weeks or months. Unfortunately, we do not know the amount of free CaO that was present in the samples examined. Our calculations show that if they had been burned to equilibrium, no lime would remain uncombined. But free lime was present, in some cases at least, as indicated by the soundness test, the phenol test and expansion in water.

The experiments reported indicate also that free MgO, in amounts from 4.2 to 7.3 per cent, may cause an expansion of a few tenths of one per cent at 3 to 10 years immersion in water. It is possible of course that these values may be inaccurate as the calculations assume a condition of equilibrium. It is of interest to observe, however, that a calculation made on the 32 cements of the investigation reported upon this year by the Society's Committee C-1 on Cement[1] shows an average free MgO content of only 1.2 per cent, and not one of the 32 contains as much as 4.2 per cent, which is the lowest value in the samples reported by Mr. White.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 233 (1928).

Mr. Bogue. To summarize, I wish again to commend Mr. White for the very interesting and useful data he has obtained on his long-time tests. They show very nicely the possible expansive effects due to large excess of free CaO or free MgO. It may be urged however that these excesses in both cases are probably greater than are present in the modern commercial portland cements.

The conclusion of Mr. White, that the amount of magnesia that will be found in the product in uncombined form is determined by the percentage of lime present, is shown on the basis of recent investigations not to apply for compositions approaching that of portland cement. The free MgO will decrease however as Fe_2O_3 increases in the mixture.

Mr. Brown. MR. H. E. BROWN[1] (*presented in written form*).—All those who are interested in cement should accord to this paper by Mr. White a scientific importance, the measure of which bears a direct relation to the time covered by the investigation. There have been many other investigations in the past concerning volume changes in cements, but even those of the Germans have not exceeded 10-year periods on laboratory controls and have not included such a range in a continuous undertaking.

The construction difficulties, and sometimes dangers, due to expansion in cement are so apparent in practice and so clearly shown in this investigation that the suggested modifications in manufacture and use should receive an attentive hearing. The excuse frequently used by so-called technicians and some manufacturers that conclusions should not be drawn from a "few results on bars and briquets" does not invalidate the evidence of which the findings in this paper are a part, that concrete of greater permanence is a prime necessity.

About 16 years ago I presented a paper at The National Cement Users Association Meeting[2] in which the characteristics of so-called iron-ore cement were pointed out. Comparative tests were made on several brands of selected excellence of portland cement with cements of low alumina, high ferric oxide, relatively low lime and magnesia. Accelerated tests under concentrated sea-water attack were made on briquets of neat and 1:3 mortars as well as upon bars sawed from the neat cement briquets. Without exception the disintegrating effects were less on these iron-ore cements than on the standard portland cement; there was a wide variation in the portland cement not accounted for by differences in chemical composition.

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[1] Consulting Chemical Engineer, New York City.

[2] See *Proceedings*, National Cement Users Assn., pp. 578–596 (1912).

Mr. Brown.

In Mr. White's investigations, conditions of expansion appear to be purposely chosen to run parallel with probable conditions. If expansion and disintegration are related, there is found to exist a close similarity in the disclosed results in this paper and those reported on iron-ore cements as shown in my paper of 16 years ago.

In Feichtinger's *Chemische Technologie* published in 1885, pages 285 to 287 are tables showing the expansion of eight brands of cement, tests being made on neat cements and 1:3 mortars over a period of a year. On both neat cements and mortars the variation between high and low was 370 per cent. High expansion on neat cement gave also a high expansion on the 1:3 prisms.

The differences between any eight brands of to-day's manufacture would, without doubt, show as much variation. From a study of the behavior of cements 131*E* and 131*G* given in Mr. White's paper it is shown that on one year's test only a small difference existed at the end of one year, while at the end of 20 years the differences were of the order of 350 per cent. These large differences make calculations on actual structures most difficult and indicate the necessity of considerable guesswork.

In this connection, where conditions are produced that simulate weather changes on hard roads in most sections in the United States the changes then mount to 600 per cent over those where fairly constant conditions are maintained.

Feichtinger shows that 1:3 mortars have an expansion of close to 25 per cent of that of neat cement; taking these facts as a basis of calculation, ample reason is found to exist for the rather unsightly appearance of a too large number of concrete structures which have been exposed to weathering and consequent expansion, especially on those of upwards of 10 years of age.

It is evident from a study of the curve for 146 *A-IV* that its expansion is not yet completed.

Mr. N. C. Johnson has demonstrated[1] by photographs of sections cut from concrete structures of long standing what relatively large quantities of cement still remained unhydrated and incidentally that the principal barriers to complete hydration are the surface tension of water and the neutral walls of calcium hydrate or carbonate crystals that prevent penetration of moisture. From this it follows that some of the several low-cost salts which would lower surface tension and at the same time prevent drying might be more generally used; also that a lowering of the calcium content of the cement is desirable.

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[1] *Transactions*, Am. Soc. Mechanical Engrs. (1915).

Mr. Brown. Colloidal silica raises no similar barrier to hydration as does calcium hydroxide; here again we come upon a confirmation of the work of Mr. Michaelis of some forty years ago who vigorously contended for low lime and high content of gelatinous silica in hydraulic cements.

Some years ago a theory was propounded that a protection was afforded concrete surfaces by the skin of calcium carbonate or basic carbonates produced by the carbonation of the free calcium hydroxide in set cement. Just a little observation of the extreme weathering effects occurring on exposed limestone surfaces should convince anyone of the futility of such claims when permanency is required.

There is considerable evidence that the cement manufacturers are becoming aware of the demands for better, more permanent cements. This is shown by finer grinding of the raw and finished product, twice-burned cements and the incorporation of compounds designed to lower the surface tension of mixing water. The engineering profession has not as yet been able to agree upon the value of accelerated tests for judging innovations of manufacture. Therefore, it is left too much in the dark as to what twenty years of exposure will do to concrete structures.

Mr. Bateman. MR. JOHN H. BATEMAN[1] (*by letter*).—The phase of Mr. White's work which dealt with the effect of free lime on volume changes of portland cement is particularly interesting. The expansion of bar 78 *B* which was exposed to air illustrates the danger of using unsound cement.

The writer has had a great deal of experience with unsound cement and it is his opinion that unsoundness is most commonly due to free or loosely-combined lime. The importance of the soundness determination of portland cement is not fully appreciated by users of this material, probably for three reasons: first, because of the frequent lack of agreement between laboratories in regard to the results of the steam test for soundness; second, because of the effect of aging of the cement, or the clinker, which causes hydration of the free lime; and, third, because of the lack of agreement between laboratory results and actual use.

Some of the reasons for disagreement among laboratories in the results of the steam test for soundness were determined several years ago by the U. S. Bureau of Standards.[2] The principal reason, perhaps, is due to the length of time the soundness specimen is stored in moist air prior to placing it in the 5-hour steam bath required by the stand-

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[1] Research Professor of Highway Engineering, Louisiana State University, Baton Rouge, La.

[2] J. R. Dwyer, "Effect of Age of Test Pieces in Soundness Tests of Portland Cement," *Concrete* (Cement Mill Edition), December, 1920, p. 129.

Mr. Bateman.

ard method. The standard procedure requires moist-air storage of the specimen for 24 hours before placing in the steam bath. The writer has found that it is not an uncommon practice in cement-mill laboratories to store soundness specimens in moist air for such short periods as 10 to 12 hours. Contrary to the general belief among many chemists, this is often a much less severe test than the standard time of 24 hours. This fact has been demonstrated many times by the writer.

A few years ago the writer rejected a large consignment of portland cement which was found to be unsound. As a precaution, samples of this cement were submitted to the U. S. Bureau of Standards and to the U. S. Bureau of Public Roads. These laboratories confirmed the writer's determinations. The cement was finally accepted on the basis of satisfactory soundness results obtained by some laboratory after efforts to have the writer change his reports had failed. It was significant that short-time tests by the writer indicated the cement was sound. The fact that the soundness test may be manipulated so as to produce a satisfactory result with unsound cement affords an excellent opportunity for an unscrupulous manufacturer to cause a user who is unfamiliar with these facts to discredit the results of a reputable and conscientious laboratory operator. Mr. White several years ago described a microscopic method[1] for detecting free lime in portland cement. The case of unsoundness just described was confirmed by this method.

When unsoundness of cement is due to free lime the condition may be overcome by storage under proper conditions. The moisture present in the atmosphere causes the lime to hydrate and become relatively inert. The cement must be stored in such a way that the air has an opportunity to circulate freely around each sack. The time necessary for hydration of the lime will depend upon the amount of atmospheric moisture, the temperature of the air, and amount of free lime present, as well as upon the opportunity of the air to come in contact with the cement. The writer has frequently observed instances of recovery in periods of 24 to 72 hours in the case of small samples. The effect of air storage was strikingly illustrated to the writer a few years ago in a case involving a carload of cement which the writer had found unsound and which had been reported sound by the producing company's laboratory. The car of cement was resampled, the new samples being taken by an employee of the cement company. At the suggestion of the writer two separate

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[1] Alfred H. White, "Free Lime in Portland Cement," *Journal of Industrial and Engineering Chemistry*, Vol. I, No. 1, January, 1909.

Mr. Bateman. samples were secured, one from several sacks in the top layer of sacks in the car, and the other from several sacks in the lowest layer (next to the floor) in the car. These two samples were then divided, one-half going to the cement company and the other half to the writer. The sample from the top of the car was found sound by the writer but the sample taken from the lower tier of sacks in the car was found to be unsound. Unfortunately the two samples were mixed together in the laboratory of the cement company; however, the mixed sample when submitted to the standard method was unsound. This instance illustrated not only the importance of adhering to the proper time of storage but also the fact that aeration had caused hydration of the free lime in the top sacks whereas in the lower sacks this opportunity for hydration had not been present.

The injurious effect of unsound cement is not always apparent. The writer is familiar with several instances where concrete structures made with unsound cement have failed; he is also familiar with numerous instances where no destructive effects were apparent. The reasons for this lack of agreement between laboratory results and actual use were explained twenty-five years ago in a paper[1] before this Society and it is unnecessary to repeat them here.

Mr. Davis. MR. R. E. DAVIS.[2]—I take this opportunity to compliment Mr. White on the thoroughness with which he has gone into the subject of volume changes in cement, extending over this long period of time.

At the Materials Testing Laboratory of the University of California we have had in progress, during the past few years, an investigation somewhat paralleling that of Mr. White, but on larger specimens and upon concretes and mortars rather than upon cements. At the present time we have approximately 350 3 by 3 by 40-in. specimens under observation. We have gone to considerable trouble to secure constant conditions with regard to storage, constructing rooms in which we can maintain any desired humidity and temperature. Last September I presented a paper before the International Congress for Testing Materials[3] giving the results of our studies up to that time. In these experiments we have sought to determine the effect not only of the richness of mix but also the effect of the quantity of mixing water, the character of aggregates, and the gradation of aggregates.

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[1] W. P. Taylor, "Soundness Tests of Portland Cement," *Proceedings*, Am. Soc. Testing Mats., Vol. III, p. 374 (1903).

[2] Professor of Civil Engineering in Charge of Mats. Testing Laboratory, College of Civil Engineering, University of California, Berkeley, Calif.

[3] "Volumetric Changes in Portland-Cement Mortars and Concretes Due to Changes Other than Variations in Temperature," *Proceedings*, Internat. Congress Testing Mats., Amsterdam, Part II, p. 145 (1927).

Mr. Davis.

Also studies have been made to determine the effect of various humidities upon the rapidity with which the shrinkage takes place when in air, and to determine the effect of alternate wetting and drying for periods of time varying from a minimum of five days for one group, to a maximum of six months for another group.

We are finding that the character of aggregate and the gradation of aggregate, as well as the quantity of cement, has an important bearing upon the volume changes; also that the quantity of mixing water materially affects the shrinkage when in air and the swelling when in water. In general, other things being equal, the shrinkage of a concrete when allowed to dry in air, seems to vary with the surface modulus of the aggregate. The shrinkage is decidedly different for a granitic aggregate than one composed of ordinary gravels. In tests using granitic aggregates, where as much as 26 per cent of fine aggregate passed the No. 100 sieve, the shrinkage was not materially different for concretes in which all fines were included than it was for concretes from which all material passing the No. 100 sieve was excluded. But when using approximately the same gradation with a natural gravel, there was a very marked difference, the shrinkage for those concretes in which the fines were included being much greater than that for concretes from which the fines had been excluded. Also it has been observed that for the same gradation the concretes from granitic aggregate consistently show much smaller shrinkages than do concretes from gravel.

We are also making a series of tests on cement mortars in which various admixtures are employed. One of the curious things we have discovered is that the behavior of a given mortar, when cast in the form of a bar, is no criterion to the behavior of that same mortar when used for the laying of brick, as in a brick pier. Without exception, the brick pier instead of shrinking when in a dry atmosphere, swells for a considerable period of time. Perhaps the explanation is as follows: The bricks are not fully saturated when they are laid. Nearly as soon as placed they absorb the excess moisture from the mortar, so that we have, in effect, a very dry mortar. With the passage of time, as the hydration of cement goes on and calls for more moisture, that moisture is transferred from the brick back into the mortar and the colloids surrounding the cement grains swell. Observations show that in dry air this swelling may continue for six months or even longer before there is indication of a shrinkage. With the passage of time the volume thereafter decreases but, generally speaking, when a state of volumetric equilibrium in dry air is reached the volume is not less than that of the pier when first constructed.

Mr. Davis. In closing I should like to voice the opinion that engineers are beginning to appreciate the necessity of a knowledge of volumetric changes for the proper design of concrete structures. More than once during the past few years I have observed where a lack of knowledge on the part of the designing engineer as to what was actually going on in concrete when with the passage of time it is allowed to dry or when it becomes water soaked, has actually resulted in failures, and I have a feeling that perhaps the largest contributing factor to the slow destruction of some of our concrete structures to-day is the volumetric change that is continually taking place throughout the life of the concrete.

Mr. Miner. MR. J. L. MINER.[1]—I should like to add one more thought to the discussion of Mr. White's very interesting paper. Mr. White and Mr. Bogue have brought out some of the factors that influence volume changes. It seems to me that there is still another factor, one which Mr. White referred to rather incidentally—it is the character of the products of hydration.

Mr. D. G. Miller, in his investigation of the action of sulfate water on concrete, and his results have been largely substantiated by Thorvaldson of the University of Saskatchewan, has shown that if mortar or concrete after twenty-four hours curing in moist air at 70° F. is subjected to water vapor at temperatures of 212° F. and higher, the resistance of the mortar or concrete to the action of sulfate water is greatly increased. This resistance of the concrete is not so appreciably increased when the concrete is cured in water vapor at lower temperatures. It would seem that this water vapor treatment of the concrete at temperatures of 212° F. and higher affects the character of the products of hydration. It will be interesting to learn whether this water vapor treatment also reduces the tendency of concrete to expand or contract with changes of moisture content.

Mr. Bates. MR. P. H. BATES.[2]—Mr. Brown commented particularly on iron-ore cement. Apparently there is an actual manufacturing difficulty in making such cement. Those who have attempted it have discontinued it. I may say that two iron-ore cements were used in the largest series of sea-water tests ever made, carried on by the U. S. Geological Survey and the Bureau of Standards at Atlantic City, N. J. These cements stood up fairly well in these tests.

When we come to the matter of composition and the effect of expansion or disintegration due to composition, then I am in accord with Mr. Miller's contention that composition is not at all indicative

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[1] Technical Manager, The Atlas Lumnite Cement Co., New York City.

[2] Chief, Clay and Silicate Products Division, U. S. Bureau of Standards, Washington, D. C.

Mr. Bates.

of disintegration.[1] Mr. Miller attempts to bring out that while one can make certain generalizations there is always the outstanding exception that kills the generalization. That is practically true from Mr. Miller's work, where he used a larger variety of cements of different compositions. Unfortunately, Mr. White had a small number of compositions but a rather large number of specimens, and therefore his generalizations may not be true. Had he, as Mr. Miller has done, used a larger number of compositions and possibly a fewer number of specimens, his conclusions would have been more valid.

Mr. White.

MR. A. H. WHITE[2] (*author's closure by letter*).—The value of the equilibrium studies which are being made under Mr. Bogue's direction is fully appreciated, but it has yet to be shown that his calculated values for free magnesia apply to commercial cements. A sure confirmation can only be gained by measurements of volume changes carried out over long periods. The consequences of free magnesia may be so serious that it would seem that no efforts should be spared to determine the conditions under which it may manifest itself in commercial cements.

Experiments such as Mr. Davis is conducting are not only valuable now, but will be of increasing value as years pass.

Perhaps we may at some future date have actual data on volume changes from the laboratories which Mr. Bates and Mr. Bogue direct.

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[1] D. G. Miller, "Resistance of Portland-Cement Concrete to the Action of Sulfate Waters as Influenced by the Cement," see p. 448.

[2] Professor of Chemical Engineering, University of Michigan, Ann Arbor, Mich.

A PLASTIC MORTAR COMPRESSION TEST FOR CEMENT

By Edward M. Brickett[1]

Synopsis

One of the chief functions of the Central Laboratory of the Lehigh Portland Cement Co. is the making of a systematic study of the quality of cements produced at the company's plants. The quality studies are based on carefully controlled compression tests of concrete. The program is so arranged as to afford an opportunity for study of other types of tests, paralleling the concrete tests, for the purpose of developing a simpler and more economical method of determining the concrete-making qualities of cements. The standard Ottawa sand mortar tension test has been tried; so also has the compression test of a fluid cement-water mixture proposed by D. A. Abrams. Neither test has indicated concrete-making quality to a satisfactory degree. A compression test of a plastic mortar gaged to the same water-cement ratio as the concrete has shown very encouraging results and seems to offer potent possibilities as a specification test which will give a much closer indication of the concrete-making qualities of cement than does the present tension test of standard Ottawa sand mortar.

The problem of devising a specification test for portland cement which will give a true indication of concrete-making qualities is ever assuming more importance. The results reported in this paper are offered as furnishing the most promising basis found so far for a solution of the problem.

Introduction

For the past two years the Central Laboratory of the Lehigh Portland Cement Co. has maintained a continuous research program involving quality studies of cement. These studies have been based on very carefully controlled compression tests of concrete made from samples of cement taken at regular intervals at each of the company's plants. In addition to the concrete studies, other tests have been conducted concurrently on the same cement samples in an endeavor to find a test which would accurately indicate concrete-making quality. During the first twelve months of operation two types of indicator test were studied: the standard Ottawa sand mortar tension test as prescribed in the A.S.T.M. Standard Specifications and Tests for Portland Cement (C 9 – 26)[2] and compression tests of

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[1] Engineer of Tests, Research Division, Lehigh Portland Cement Co., Allentown, Pa.

[2] 1927 Book of A.S.T.M. Standards, Part II, p. 23.

2-in. cubes of a fluid cement-water mixture proposed by D. A. Abrams.[1] These cement-water mixtures were gaged to a common flow; the water content averaged approximately 39 per cent of the cement by weight. A study of the results obtained over the twelve months showed conclusively that the fluid cement test was not a satisfactory indicator of the concrete-making qualities of a cement—in fact, that it was not as good as the present tension test in this respect. The results also showed that the tension test was far from being a satisfactory indicator.

PLASTIC MORTAR TEST

In planning the work for the second twelve months of research, a change in the program of tests seemed desirable. Since the fluid cement-water mixture had been given a thorough trial and had been found wanting, it was felt that further work with it was not warranted. However, experience with the fluid cement test suggested a modification of it based on the same principle and eventually led to the plastic mortar test with which this paper deals. A plausible explanation of the failure of the fluid cement-water-mixture tests to indicate concrete strengths lay in the great difference between the water-cement ratio in the mixtures and in the concretes with which they were compared.

The fluid cement-water mixtures had a water-cement ratio of about 0.6; the concretes had a water-cement ratio of about 1.00. It does not seem likely that all cements have the same strength relations one to another at both of these water-cement ratios. It seems much more probable that the water-cement ratio-strength curves for cements bear varying relations to one another, that some pairs of curves converge as they pass from one water content to the other, that some diverge, and that some even cross. Such a state of affairs would account for the lack of concordance that has been observed between the results of fluid cement tests and concrete tests of the same cements.

The logical step then was to devise a test using a water-cement ratio of approximately 1.00, thus duplicating the concrete in water content. A fluid cement-water mixture of such water content would be so fluid that segregation would be inevitable in the freshly poured specimens. The problem was to employ a water-cement ratio of 1.00 and to use a mixture of such consistency that segregation or settlement in the molds would be overcome. The introduction of a graded sand to the cement-water mix was the next logical step. A few trials showed that a satisfactory mortar could be compounded with the desired water-cement ratio and apparently made stiff enough to resist segregation in the molds and yet so plastic as to minimize the personal factor of puddling or rodding the specimens. The procedure which has been followed in actual practice has been to use a mortar of 1 part of cement to 2.75 parts of sand, by weight, gaged to the same water-cement ratio as the concrete which was previously made from the same cement sample. The results obtained so far show that in its ability to indicate concrete-making quality this plastic mortar test is better than any other test which has been proposed or used to date.

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[1] "Tests of Fluid Cement-Water Mixtures," Appendix to Report of Committee C-1, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 307 (1927).

METHODS OF TESTING

The concrete strengths used as the basis for comparison were obtained through closely controlled tests. This control is the result of a carefully developed and organized routine covering materials, methods and conditions. The main characteristics of the concrete tests are outlined below:

Test Pieces.—3 by 6-in. cylinders.

Proportions.—1 part cement, 2.04 parts sand, and 3.04 parts pebbles, by volume.

Aggregates.—Delaware River Sand, 0 to No. 4; Delaware River Pebbles, No. 4 to ¾ in.; Fineness Modulus, 5.06 ±0.05; Weight per cubic foot, 121 lb.; Colorimetric Value, 0. The sand and pebbles are separated by sieving into seven distinct sizes which are then recombined by weight to insure constant grading of the concrete aggregate.

Mixing.—Each batch is sufficient for ten specimens. Mixing, by hand, is continued for 2 minutes and 20 seconds after the introduction of the water. Concrete is then placed in the flow mold and the flow test is started exactly 3 minutes after the introduction of the water.

Consistency.—80-per-cent Flow (About 6½-in. slump).

Water Ratio.—Varies slightly according to consistency, but is approximately 1.00 or 7½ gal. of water per sack of cement, disregarding absorption by the aggregate.

Companion Specimens.—Each concrete value used is derived by averaging the results found for three companion specimens. These companion specimens are made on different days by different operators, so that the effects of personal factor and temperature and humidity are generalized.

Curing.—Specimens stand in the mixing room for 4 hours, then are removed to a damp room where they are capped and stored at 70° F. and 100 per cent Relative Humidity until tested.

Ages at Test.—Six ages as follows: 1 day, 2 days, 4 days, 7 days, 14 days and 28 days.

All tension tests were made in strict accordance with the present A.S.T.M. Standard Specifications and Tests for Portland Cement

(C 9 – 26) with additional specimens for tension tests at 1 day and 3 days. All the work was done by one experienced operator and we have reason to believe that the results approach the practical ultimate in reliability for this type of test.

The fluid cement-water mixtures were made in accordance with the methods and conditions proposed by D. A. Abrams.[1] The water content was governed by a flow test. A flow mold with a capacity of 400 cc. of mixture and with an orifice at the bottom was held above a level glass plate so that the orifice was just 10 cm. above the plate. A trap door in the bottom of the flow mold permitted the flow of the mixture through the orifice on to the glass plate where the 400 cc. of mixture formed a "pan-cake" whose diameter could be measured. The mixtures were gaged to a common consistency such that the "pan-cake" had a diameter of 250 mm. This consistency is similar to that of heavy cream. Two-inch cubes were poured with this fluid cement-water mixture. Pouring was done in the damp room and the specimens were stored there until tested.

The plastic mortar mixes were made with 1 part of cement to 2.75 parts of sand by weight. The sand used was that portion of the concrete sand finer than No. 8. To insure constant grading the sand was fabricated from four sieved sizes of sand. The result was a graded sand with a fineness modulus of 2.23 ± 0.05. The amount of water to be used was determined from the previously made concrete and was calculated to give the same water-cement ratio as used in the concrete, disregarding absorption by the aggregate. The mortar was mixed with an iron spoon in an 11-in. iron stew pan; mixing was continued for 2 minutes and 30 seconds after the introduction of water. Two mortar batches, each sufficient for five 2-in. cubes, were necessary in making a set of ten test pieces. The two batches were mixed and poured in rotation, using identical proportions in fabricating each batch. The cube molds were filled in two layers, each layer being rodded 25 times with a $\frac{1}{4}$-in. rod. A surplus of mortar was left on the mold and was struck off two hours after pouring.

All molding of the plastic mortar cubes was done in the damp room where the temperature has never varied more than 3° F. either way from 70° F. and the relative humidity has been found repeatedly as 100 per cent by periodical tests with a sling psychrometer. The mortar cubes remained in the damp room until time of test. All cube specimens were tested on their sides, two of the molded faces acting as the bearing surfaces. A sufficient number of cubes were made to provide a parallel specimen for every concrete cylinder. Thus, every plastic mortar value used is derived by averaging the results for three companion specimens. The three companion specimens in every case were made on different days and by different operators to generalize any personal factor that might exist.

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[1] "Tests of Fluid Cement-Water Mixtures," Appendix to Report of Committee C-1, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 307 (1927).

PRESENTATION OF DATA

Due to the differences in absolute strengths of the concrete cylinders and the various indicator test pieces, it became necessary to develop a system of "relative strengths" in order that the several tests might be fairly compared. This was done by determining the average strength for all of the cement samples as revealed by any type of test, concrete, fluid cement, standard mortar, or plastic mortar, at a given age, calling that 100, and then expressing the corresponding strengths of individual cements as percentages of that average. For example, if the average concrete strength for all cements at 14 days were 2000 lb. per sq. in. and a certain cement developed a concrete strength of only 1800 lb. per sq. in. then the concrete relative strength of that cement at that age would be 90. The several indicator tests could be exemplified in the same way. To judge the value of any indicator test, therefore, for its ability to predict concrete-making

TABLE I.—DIFFERENCES IN AVERAGE RELATIVE STRENGTHS.

Differences in average relative strengths between concrete and fluid cement cubes, concrete and standard mortar briquets, and concrete and plastic mortar cubes.

Mill	Fluid Cement minus Concrete							Standard Mortar minus Concrete					Plastic Mortar minus Concrete						
	1 day	2 days	4 days	7 days	14 days	28 days	Average	1 day	3 days	7 days	28 days	Average	1 day	2 days	4 days	7 days	14 days	28 days	Average
1	−6	−15	−14	−20	−22	−19	−16	−9	−19	−15	−10	−13	−6	+1	+3	+2	+2	0	0
2	−29	−23	−26	−25	−24	−10	−23	−9	−8	−6	−7	−8	−10	−5	−6	−5	−9	−9	−7
3	−11	−11	−10	−10	+6	+9	−4	+6	−10	−7	−3	−4	+3	−2	+1	+9	+6	+2	+3
4	+11	+4	+3	+4	+3	+1	+4	−4	−5	−8	−3	−5	+11	+6	+2	+3	+4	+1	+4
5	−1	−4	−12	−9	−14	−9	−8	+8	0	−11	−7	−2	+8	−8	−11	−2	+2	+5	−1
6	−1	+5	−2	0	0	+3	+1	+7	−1	−10	−5	−2	−4	−1	0	−1	+3	+4	0
7	−9	−13	−14	−12	−12	−7	−11	−1	0	−11	−5	−4	+8	+4	+4	+4	+4	+2	+4
8	+3	+2	+1	+2	−5	−5	0	−12	−2	+3	+2	−2	−1	0	+5	−1	+1	+4	+1
9	+1	+5	+7	+4	+9	+7	+6	−28	−10	−3	+6	−6	−2	+1	−4	−1	+3	−4	−1
10	−9	−8	−12	−9	−11	−13	−10	+11	+8	+2	−5	+4	+1	+1	−4	−5	−5	−1	−2
11	−4	+1	0	+2	−2	0	0	−5	+3	+5	+3	+2	+5	+1	+2	+2	−1	+2	+2
12	0	0	+2	+6	−1	−5	0	+16	+12	+4	−1	+8	−2	−6	−1	+2	+3	−2	−1
13	+11	+12	+15	+13	+8	+1	+10	−1	0	+6	+5	+2	−3	−2	−2	−1	−2	0	−2
14	+10	+13	+14	+9	+6	0	+9	+11	+11	+17	+17	+14	+1	−1	+1	−1	−1	0	0
15	+15	+8	+12	+10	+8	+4	+10	+18	+8	+9	0	+9	−1	−1	−1	−3	−4	−3	−2
16	+6	+8	+5	+7	+16	+14	+9	−1	+4	+4	+4	+3	−4	+2	−1	0	0	−1	−1
17	+16	+22	+26	+22	+22	+16	+21	−13	+8	+16	+13	+6	+1	+6	+6	+3	+6	+7	+5
Grand Average Differences							8.4	....	....	....	....	5.5	....	....	....	....	....	....	2.1

NOTE.—The 3-day differences for the standard mortar are calculated as the difference between the 3-day standard mortar relative strength and the average of the 2-day and 4-day concrete relative strengths.

The concrete relative strength is used as the basis for the computation; the plus sign denotes the indicator test as showing a higher relative strength than did the concrete, and the negative sign denotes the indicated relative strength as lower than the concrete relative strength.

qualities, the relative strengths shown by that test may be compared with the corresponding concrete relative strengths. The more closely these relative strengths coincide the more satisfactory is the indicator test.

Table I is presented to show the differences in average relative strengths between concrete and the several indicator tests for each of seventeen mill products at each test period up to 28 days. The values are not derived from a single test; they are computed by averaging the results for monthly samples from respective mills over an extended period. The concrete - fluid cement comparisons are based on monthly tests over one year; the differences shown for the other two indicator tests are derived from the data of six months. Thus, the differences recorded cannot be attributed to the possible uncertainties of a single test, but are averages that show *real trends* for the respective mill products. For the fluid cement differences each value is derived from 36 tests; for standard mortar and plastic mortar each individual difference presented is derived from 18 tests. Attention is called to the grand average differences, which may be considered as a measure of the faithfulness of each type of indicator test.

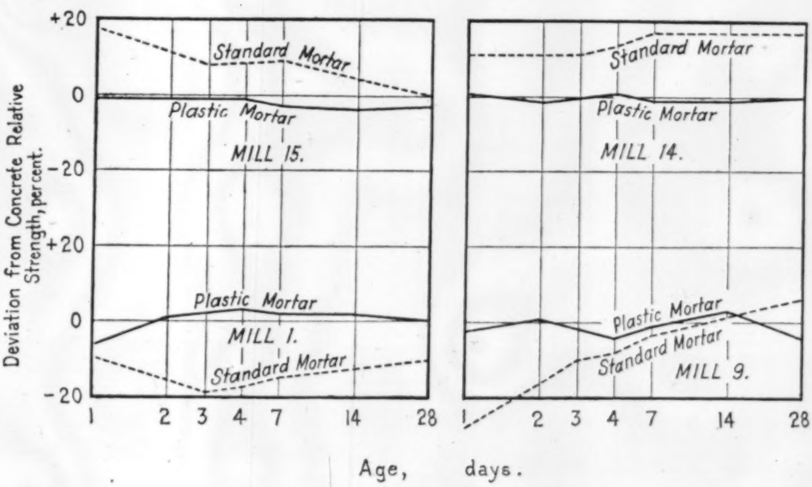

FIG. 1.—Comparison of Relative Strengths of Cements in Standard Mortar (Tension) and Plastic Mortar (Compression) with Concrete Relative Strengths.

In Fig. 1 relative strength curves have been prepared which show the deviation of tensile strengths and plastic-mortar strengths from the concrete strengths for four different mill products at the several test periods up to 28 days. The data for these curves were taken directly from Table I. Each set of curves was chosen to exemplify a characteristic misinterpretation of cement quality which may be encountered with the standard tension test. For example, the tension tests for Mill 1 indicate a lower concrete quality than actually exists, while for Mill 9 the tension tests show a higher rate of strength gain than actually occurs in the concrete. Attention is called to the favorable indications of the plastic mortar test in every case.

In Fig. 2 a slightly different scheme of comparison is used. By averaging the relative strengths at all ages up to 28 days a figure is obtained which is termed "relative strength level." Here again, if an indicator test functions perfectly it should rate any mill product at the same relative strength level as does the concrete test. The mill products are arranged in order of descending concrete relative strength levels. The fluid cement-water mixture values are averages for performance over twelve months, the plastic mortar and standard tension mortar values are averaged for six months.

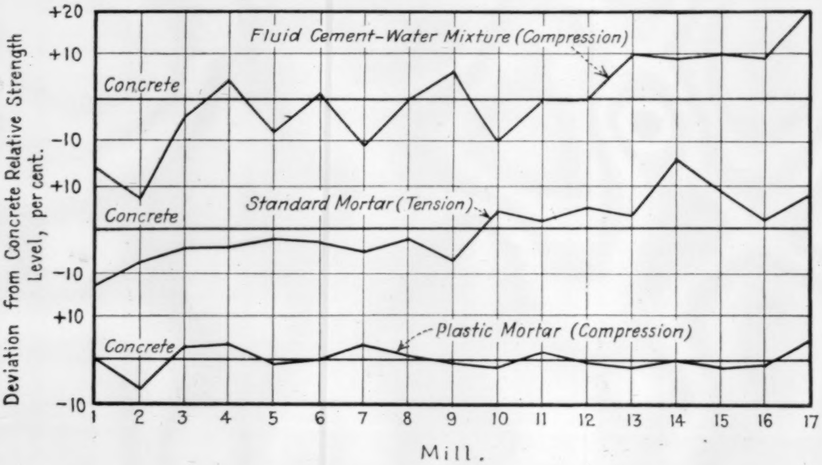

FIG. 2.—Comparison of Relative Strength Levels of Cements in Three Types of Indicator Test with Concrete Relative Strength Levels. The Mills are Arranged in Order of Descending Concrete Relative Strength Levels.

A detailed discussion of these curves seems unnecessary. The general slopes of the fluid-cement line and the standard-mortar line show that both of these tests tend to penalize the high-strength cements and to give too high a rating to the low-strength cements. In the case of the fluid cement-water mixtures the value shown by the indicator test differs from the actual concrete value by more than 5 per cent for eleven out of the seventeen mill products, the average difference for all is 8.4 per cent. The tension test is "off" by more than 5 per cent for six out of the seventeen mill products and the average difference for all is 5.6 per cent. The plastic-mortar test is 7 per cent low for Mill 2; in all other cases it is within 5 per cent of the concrete value and the average difference for all is 2.1 per cent.

Table II is presented to show the ratios of the absolute strengths in each type of test to the absolute concrete strength. It may be noted that the plastic mortar ratios are close to unity, so close as to assume a very important practical significance.

TABLE II.—RATIOS OF ABSOLUTE STRENGTHS.

Ratios of the absolute strengths of fluid cement compression cubes, standard mortar tension briquets and plastic mortar compression cubes to the absolute strengths of the corresponding concrete compression cylinders at the several test periods.

AGE, DAYS	FLUID CEMENT / CONCRETE	STANDARD MORTAR / CONCRETE	PLASTIC MORTAR / CONCRETE
1	3.36	0.43	1.03
2	3.16	....	1.05
3	....	0.28[a]	....
4	3.06	....	1.10
7	3.22	0.19	1.12
14	3.08	....	1.13
28	2.77	0.13	1.14

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[a] The ratio noted for the 3-day standard mortar strength is the ratio of the 3-day standard mortar strength to the average of the 2-day and 4-day concrete strength.

TABLE III.—AVERAGE VARIATIONS OF COMPANION PLASTIC MORTAR SPECIMENS FROM THEIR OWN AVERAGE.

The variations shown are averages for all mill products tested during two different months. Variations are expressed as a percentage of the average.

MONTH	1 DAY	2 DAYS	4 DAYS	8 DAYS	14 DAYS	28 DAYS	AVERAGE
September, 1927	5.2	4.7	5.6	5.1	5.4	3.6	4.9
October, 1927	5.4	4.7	3.9	4.1	5.3	4.7	4.7
Average	5.3	4.7	4.8	4.6	5.4	4.2	4.8

In describing the method of test it was stated that the plastic mortar relative strengths were computed by averaging the results for three companion specimens which were made by different operators and on different days. Table III shows the average variations of the three companion specimens from their own average for each age and for all mill products tested in two different months.

DISCUSSION OF RESULTS

There are certain cardinal features which should always be incorporated, if possible, in devising any specification test. These features are: (1) faithfulness in indicating quality; (2) minimum variation in test results for companion specimens or repeat tests; (3) freedom from sensitiveness to the personal factor or other secondary influences; (4) simplicity in all of the operations of testing; and (5) low cost.

Examination of Figs. 1 and 2 and Table I show the superiority of the plastic mortar test in faithfully indicating the concrete-making qualities of cement. Figure 1 and Table I show studies of the accuracy of the indicator tests at each test period. Figure 2 compares the indicator tests in a different manner and brings out the same conclusion as to the superiority of the plastic mortar test as an indicator of the concrete-making qualities of cement. The average deviations of Fig. 2 and the grand average differences of Table I are dependable indications of the accuracy of each type of test, as they are based on a large amount of data derived from carefully controlled tests extending over a considerable period of time.

The plastic nature of the mortar used in this indicator test is such as to reduce to a practical minimum the personal factor of rodding or puddling the cube specimens. The sand is graded to enhance plasticity and the mix is wet enough to insure easy working in the molds. In studying the variations among companion specimens (Table III) it should be remembered that the companion specimens were made on different days and by different operators. The variations at 1 day are comparatively high, as might be expected, and yet the grand average variation of 4.8 per cent indicates very good agreement. In the report of cooperative tests of 32 cements by 47 laboratories, conducted last year by Committee C-1 on Cement of this Society, a weighted average variation of 3.8 per cent was found among companion tension briquets, but these companion specimens were made from a single batch by a single operator on the same day so that the conditions were far more favorable for small variations.

The fabrication of plastic mortar cube test pieces is simple; the procedure closely parallels that of the fabrication of concrete specimens. The cube test piece offers added simplicity in that no capping is necessary, since two of the molded sides are used as bearing surfaces during testing. The specimens are tested in a compression machine; the operation of centering the specimen in the testing machine requires care but is simple with suitable equipment.

The principle of the plastic mortar test is based chiefly upon the use of a water-cement ratio identical with that of the accompanying concrete. In these pioneer tests it was necessary to use a single sand of carefully controlled gradation in order to be certain that any irregularities in results which might occur were not due to variations in sand. If water-cement ratio is the governing factor in this test, the possibility exists that satisfactory and comparable results may be obtained by widely separated laboratories using clean sands of local origin graded within certain limits, by the mere use of equivalent water contents. If this is true, the necessity for a single standard sand in cement testing would be removed.

The absolute strengths of the plastic mortar cubes agree fairly closely with the concrete strengths, as may be seen by Table II. In fact, the differences recorded might be attributable to the difference in size and shape of the two types of test pieces. The practical significance of this feature of the plastic mortar test should not be overlooked. The agreement in absolute strength between plastic mortar and concrete is so close that the plastic mortar test offers a rough means of making a direct determination of the concrete-making qualities of any cement.

CONCLUSIONS

The plastic mortar test has several desirable features and offers some interesting possibilities as a specification test for cements to replace the present tension test. The plastic mortar test has all of the advantages that have been ascribed to the fluid cement test and one or two advantages which the latter does not have. They may be summed up as follows:

1. It gives a more nearly correct measure of the concrete-making qualities of cement than does any other indicator test used or proposed to date.

2. The plasticity of the mortar minimizes the effects of personal factor of operators and of minor variations in technique.

3. Specimens are more easily made than in the case of standard Ottawa sand mortar because the mix is plastic and more easily worked.

4. It may, though it has not yet been shown that it surely will, obviate the necessity for a single standard sand.

5. It offers an entirely feasible means for making early strength tests. With good control, tests at 24 hours have been shown to be in close agreement with concrete tests at that age.

6. The absolute strength of the plastic mortar is fairly close to that of the concrete. In the tests described here the differences in absolute strength could be ascribed to differences in size and shape of specimen: 3 by 6-in. cylinders for concrete and 2-in. cubes for the plastic mortar.

REMARKS

It appears that the principle of the plastic mortar test might easily be adapted to the solution of certain concrete design and control problems. It might be applied to a strength test for a rating of concrete sands.

It is not felt at all that the test is perfected as yet. Studies must be made of the possibility of using various sands rather than a single one, of the effects on strength of different proportions and gradings of sand, of the most suitable means of prescribing the amount of gaging water to be used if this test should be developed as a specification test for portland cement and, in that case, the strength limits which should be imposed.

Further study appears to be fully warranted in view of the results to date. The work so far has merely established the parallelism between plastic mortar and concrete made with the same cement, when both are gaged to the same water-cement ratio.

Acknowledgment.-Grateful acknowledgment is accorded Mr. J. C. Pearson for his whole-hearted support, both through advice and through cooperation, in this investigation.

DISCUSSION

Mr. Jackson.

MR. F. H. JACKSON[1] (*presented in written form*).—Mr. Brickett has described a very interesting method for studying by means of a so-called "indicator test" the effect of variations in the quality of portland cement upon the strength of concrete. The data presented indicate very clearly that the plastic mortar test as developed in the Central Laboratory of the Lehigh Portland Cement Co. is a much more reliable tool for measuring the relative concrete making properties of different cements than is either the standard 1:3 mortar briquet test or the neat fluid cement test developed by Mr. Abrams. As a means for rating cements under given standard laboratory conditions, the suggested method should prove entirely satisfactory.

As the author states, however, much additional work must be done before it can be stated definitely whether the method may be developed into a specification test to replace the present unsatisfactory briquet test.

In the first place, the writer is not at all sure that it will be feasible to use any "clean sand of local origin graded within certain limits" as suggested by the author of the paper. Experiences in the laboratory of the U. S. Bureau of Public Roads in connection with plastic mortar sand mixes have indicated that, entirely apart from grading, the character of the sand grains may influence the strength of the mortar to quite a marked degree. The writer believes that, before the plastic mortar test can even be considered as a specification test, a single standard aggregate will have to be specified. There are enough variables to cloud the results without introducting an additional one.

Again, the acid test for this as well as any other suggested specification test will not come until an attempt is made to duplicate results in a number of laboratories located in different sections of the country. The fluid cement test appeared quite promising so long as the testing was confined to a single laboratory working under given conditions. When an attempt was made to test the same cements by this method in 47 different laboratories, situated from coast to coast, the results were quite disappointing.

In connection with the use of portland cement in the construction of highways, the writer has had an excellent opportunity to observe the operation of the standard cement tests in the various state highway

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[1] Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

Mr. Jackson. and commercial testing laboratories. He is convinced that by far the greatest single cause for the variations reported by different laboratories is due to variation in temperature and humidity conditions, some of which, such as the temperature and humidity of the moist closet, may be, but frequently are not, properly controlled, and some of which, such as temperature and humidity of the laboratory air, cannot be controlled by any practical means.

The writer does not wish to discourage any attempt which may be made to develop the plastic mortar test into a specification test for portland cement. He does wish, however, to emphasize the fact that many of our troubles in connection with the present specification tests for cement are due to a lack of appreciation on the part of officials in responsible charge of cement inspection of the importance of installing and maintaining adequate facilities for controlling such vitally important factors as temperature and humidity and that until we have overcome this handicap the problem of securing comparable results will not be solved, regardless of the particular type of test employed.

Another point of interest in connection with this work is the fact that, although no actual strength values are reported in the paper, it may be assumed from the nature of the study that the "relative concrete strengths" of the products of the several mills varied considerably, even for a constant water-cement ratio. It may be of interest to note in passing that, in a series of tests recently made by the Bureau of Public Roads on eight different cements from a single local market made up into concrete at a constant water-cement ratio, with every factor except the quality of the cement carefully controlled, a variation in concrete strength at 28 days was obtained equal to the difference in strength which would theoretically be secured by a change in the water-cement ratio of 0.15, or about 1.1 gallons of water per bag of cement.

The Lehigh Portland Cement Co. is to be congratulated upon the thorough and painstaking manner in which it is studying the relative concrete-making properties of its several products.

Mr. Gonnerman. MR. H. F. GONNERMAN[1] (*presented in written form*).—The need for a test that will give a definite indication of the strength property of a cement in concrete has long been recognized. The data presented by Mr. Brickett appear to give promise of a solution of the problem. His method of using a mortar test with a definite water-cement ratio as a criterion of the value of the cement in concrete seems logical and the nearer this water-cement ratio approaches that ordinarily used in

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[1] Manager, Research Laboratory, Portland Cement Association, Chicago, Ill.

Mr. Gonnerman.

concrete the more value will the test have. The exact water-cement ratio to be used is not important provided the mortar is plastic, and the same water-cement ratio is always used. However, it is extremely important that a proper consistency be selected. The tendency for over-wet mortars and concretes to segregate is well known and in selecting a consistency this condition as well as dry, non-plastic mixtures, must be avoided. In the tests reported by Mr. Brickett the consistency of the mortar is not given other than by the statement that the mix was plastic and easily molded. If his sand were recombined to give about the same grading as ordinary concrete sands, we would expect that a 1:2.75 mix with a water-cement ratio of 1.00 would be of such a consistency that some segregation would result.

Considerable work along the same lines as Mr. Brickett's has been carried out during the past 18 months in the Research Laboratory of the Portland Cement Association. Our earlier tests were carried out in connection with the cooperative tests of Sub-Committee VII on Strength, of the Society's Committee C-1 on Cement[1] on a mortar from graded Elgin sand, and on concrete from Elgin sand and gravel. The concrete mix was 1:2.4:3.6 by weight and the mortar mix 1:2.4 by weight. Both mixes were of the same nominal water-cement ratio (0.90). The results obtained were not very satisfactory. This was believed to be largely due to the consistency of the mortar which permitted of some segregation. Other tests, made on a plastic 1:2 mortar (by weight) of water-cement ratio 0.60 using standard Ottawa sand likewise did not give the concordance desired with tests on concrete mixed to a water-cement ratio of 0.90. This led to the tests with plastic mortars of the same water-cement ratio as the concrete along the same lines as followed by Mr. Brickett.

In these more recent tests the mix was 1:2.5 by weight and the water-cement ratio 0.90. The sand was 0 to No. 14 silica sand from Ottawa, Ill., from the same source as standard Ottawa sand. It was screened and recombined in the following proportions by weight: 0 to No. 100, 15 per cent; No. 100 to No. 48, 20 per cent; No. 48 to No. 28, 25 per cent; No. 28 to No. 14, 40 per cent. This particular grading was used as it gave the densest mixture of 0 to No. 14 sand, for the above sizes. When mixed in the proportions of 1:2.5 by weight with a water-cement ratio of 0.90, the mortar was of such consistency that there was no segregation and it could be readily molded.

When compared with concrete of the same water-cement ratio the strengths obtained showed results as promising as Mr. Brickett's and lead us to believe that these methods may, when fully developed,

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 261 (1928).

Mr. Gonnerman.

give a better indication of the strength property of cement in concrete than any test thus far tried. In a few cases the test has not given the concordance desired, but it is believed that further studies will explain the reasons for this.

In reporting the mean variations from the average, Mr. Brickett reports a grand average variation of 4.8 per cent for the cube tests which includes variations resulting from different operators and from making the specimens on different days. The work done in our laboratory showed mean variations for the cube tests of 2.7 per cent and 3.0 per cent, but the companion specimens were all made the same day and by the same operator.

Results of tests on two of the cements used in our study are shown to emphasize the point brought out by Mr. Brickett regarding mills 1 and 9, where the briquet tests gave a very inaccurate indication of the strength of the cement in concrete. The results for these two cements at 28 days are given in the following table in which the close agreement between the plastic cube tests and the concrete is clearly seen for cements with the same briquet strength but having different strengths in concrete:

	TENSILE STRENGTH, (BRIQUETS) LB. PER SQ. IN.	COMPRESSIVE STRENGTH, LB. PER SQ. IN.	
		2-in. Mortar Cubes of Silica Sand	6 by 12-in. Concrete Cylinders
Cement No. 12.......	400	2560	2550
Cement No. 17.......	400	4300	4240

These data are indicative of the general results obtained on the 36 different cements studied.

The suggestion that local sands of definite grading could be used in this test is worthy of study, but our knowledge of sands and the present methods of testing sands for concrete are not far enough advanced for us to know what constitutes a structurally sound sand. A very comprehensive study of sand testing applying the water-cement ratio has been made during the last year by a number of state highway laboratories and no doubt our knowledge of the behavior of sands in mortars will be greatly increased when the data from these investigations are available.

It is to be hoped that other investigators will make further tests along the lines of those described by Mr. Brickett in order to study more fully the advantages and limitations of the method.

Mr. Brickett.

MR. EDWARD M. BRICKETT.[1]—Mr. Gonnerman mentions that the consistency of the plastic mortar mix is not described in the paper. I can correct that omission very briefly. We made a flow test on every batch of plastic mortar, using a 10-in. flow table with a $\frac{1}{8}$-in. drop. The average flow for all of the batches was 56 per cent for thirty drops. The diameter of the base of the mold into which we placed the mortar for the flow test is 100 mm. After the thirty drops of the flow table, the average diameter of the resultant pancake of mortar was 156 mm. The average increase in diameter, or the *flow*, was then, 56 per cent. It was noticed that the flows of the mortars did not correspond to the flows of the concrete.

Mr. Gonnerman, in speaking of the variations among companion specimens in some mortar tests which he made, stated that his variations were less than ours and attributed the lower variations to the fact that his companion specimens were all made on the same day. Ours were made on different days and by different operators, but more than that, the water contents of the three companion specimens varied slightly in nearly every instance. Our procedure was to make the concretes first, gaging them to a flow of 80 per cent. Different operators usually required slightly different water contents to get that flow of 80 per cent. If a little more water was required for the concrete, then this same increase was observed in the parallel mortar. So, considering that our companion specimens were susceptible to so many more variations than were those of Mr. Gonnerman, it is not at all surprising to hear that his strength variations were less than ours.

Mr. Gonnerman.

MR. GONNERMAN.—I did not mean to infer that Mr. Brickett's results were not concordant. They are extremely so. I merely wished to point out that his specimens were made in an entirely different manner from ours and yet he obtained very consistent results.

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[1] Engineer of Tests, Research Division, Lehigh Portland Cement Co., Allentown, Pa.

RESISTANCE OF PORTLAND-CEMENT CONCRETE TO THE ACTION OF SULFATE WATERS AS INFLUENCED BY THE CEMENT[1]

By Dalton G. Miller[2]

Synopsis

The paper describes tests to determine the resistance of concrete, made from 30 portland cements, to the action of sulfate waters including a solution of sodium sulfate, a solution of magnesium sulfate and the natural alkaline waters of Medicine Lake, South Dakota. The tests show that standard portland cements may vary greatly in resistance to sulfate waters; under the same exposure conditions the more resistant cements have outlived those of less resistance by as much as eight times.

Portland cements that have failed quickly in the laboratory in solutions of sodium sulfate have ordinarily displayed low resistance in the field to the action of mixed salts. The most desirable cements for concrete exposed to the action of sulfate waters are those that prove most resistant to the action both of pure salts and of mixed salts.

The standard physical tests and chemical analyses of portland cements give no indication of resistance to sulfate waters. Geological differences of raw materials may possibly be a factor.

The paper outlines an accelerated test by submerging standard cement briquets for 6 months in 1-per-cent solutions of sodium and magnesium sulfates. Until a better accelerated test is developed, this is recommended for determining the resistance of portland cements to the action of sulfate waters.

Experiments in the laboratory with cement-concrete cylinders stored in 1-per-cent solutions of sodium sulfate (Na_2SO_4) indicated a great difference in 12 standard portland cements in resistance to disintegration, as previously reported.[3] The experiments were therefore broadened to include 18 other cements and also to include check tests of cylinders made of 22 of the 30 cements and exposed to the mixed salts of the natural alkaline waters of Medicine Lake, South Dakota, 18 miles northwest of Watertown.

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[1] University of Minnesota Paper No. 779, Journal Series. This paper is the result of experiments at University Farm., St. Paul, Minn., in the Drain Tile Laboratory conducted by the Department of Agriculture of the University of Minnesota, the Department of Drainage and Waters of the State of Minnesota and the U. S. Department of Agriculture.

[2] Drainage Engineer, Bureau of Public Roads, U. S. Department of Agriculture, St. Paul, Minn.

[3] "Four Important Factors in the Manufacture of Concrete Pipe for Alkali Soils," *Transactions*, Am. Soc. Agricultural Engrs., Vol. 20, p. 180 (1926).

Analyses of water samples taken from Medicine Lake at different seasons of the year have shown a total salt content ranging between 2.34 and 4.72 per cent, consisting almost entirely of magnesium and sodium sulfates. An average of four analyses of water samples taken December 10, 1923; February 14, 1924; April 29, 1924, and July 1, 1925, are given in Table I.

TEST CYLINDERS DESCRIBED

Each series of test cylinders consisted of five batches made on different days, and nine 2 by 4-in. cylinders were made from each batch. The cylinders were mixed in the proportion of 1:3 by volume with a relative consistency of 1.00 and water ratios ranging between 0.59 and 0.64 as shown in Tables II and III. The aggregate passed all standard physical tests and was separated into screen sizes and recombined for each batch to produce a fineness modulus of 4.67. The cylinders stored in the laboratory solutions were cured 24 hours in the moist closet followed by 20 days in distilled water and at 21 days were transferred to earthenware jars containing 2.6 gallons of 1-per-cent solutions of sodium sulfate (Na_2SO_4), 20 cylinders to a jar. All solutions were changed at the end of the first week after which they were changed every 2 weeks for the first year and, following the 1-year test, were changed every 4 weeks. The cylinders in those series stored in Medicine Lake were hardened 5 weeks in air in the laboratory, following 1 day in the moist closet and 20 days in distilled water, and excepting for this air-hardening period there was no intentional difference between the cylinders stored in the laboratory solutions and in Medicine Lake. Check cylinders in all cases were stored in tap water in the laboratory tank.

TABLE I.—AVERAGE OF FOUR ANALYSES OF WATER FROM MEDICINE LAKE, SOUTH DAKOTA.

ANALYSES BY THE WATER AND BEVERAGE LABORATORY, BUREAU OF CHEMISTRY U. S. DEPARTMENT OF AGRICULTURE

RADICAL	ANALYSIS, MG. PER LITER (PARTS PER MILLION)	REACTING VALUES, PER CENT
Na (Calculated)	3 036	10.38
Ca	717	2.93
Mg	5 079	36.69
NO_3	1	0.01
Cl	509	1.01
SO_4	27 021	48.22
CO_3	88	0.30
HCO_3	313	0.46
Total	36 764	100.00

In certain of the tables and figures reference is made to the "life" of different series. By this is meant the average time in weeks required for ten 2 by 4-in. cylinders to increase in length 0.01 in. (0.25 per cent). Many tests in the laboratory, dating from 1921, have shown that such a length increase for cylinders of this type of concrete stored in sodium and magnesium-sulfate solutions is indicative of strength losses of 50 to 65 per cent. (For general discussions of this method of rating, see articles published in 1922[1] and 1924.[2]) Throughout this paper the order of arrangement of the data in all tables and figures is the same as the "life" as recorded in the last column of Table II.

TABLE II.—TESTS OF 2 BY 4-IN. CONCRETE CYLINDERS, 1:3 MIX, MADE OF PORTLAND CEMENTS FROM DIFFERENT PLANTS AND STORED IN 1-PER-CENT SOLUTIONS OF SODIUM SULFATE (Na_2SO_4).

Each result is average for 5 or 10 cylinders made on 5 different days.

Cement	Water Ratio	Absorption at 21 Days, per cent	Average of Compression Tests, lb. per sq. in.					Life, weeks
			Stored in Water			Stored in Solutions		
			7 days	28 days	1 year	1 year	2 years	
No. 40	0.60	5.6	3310	4600	6480	5310	5760	210^{g}
No. 97	0.64	6.1	2710	4610	6000	5850	4890	165^{g}
No. 79	0.62	5.9	3180	4640	6410	5520	4490	165^{g}
No. 41	0.60	5.5	3460	4790	5860	5930	4280	165^{g}
No. 42	0.62	6.3	2450	3580	6220	4900	4090	165^{g}
No. 34	0.67	6.2	3160	4970	5730	5040	4050	165^{g}
No. 35	0.60	5.9	3010	4630	6250	5070	3860	150^{g}
No. 113	0.64	6.2	1960	4240	5580	4840	3000	140^{g}
No. 37	0.60	6.3	3180	4820	5740	4600	2240	115
No. 98	0.64	5.8	3710	5380	6240	5150	2430	110^{g}
No. 102	0.62	5.6	4030	5970	5890	4780	1840	95^{g}
No. 105	0.62	5.9	3560	5450	6220	4360	1550	92
No. 39	0.60	5.8	3280	5020	6080	4790	1910	88
No. 106	0.62	6.2	2930	4810	6040	3890	0	87
No. 65	0.60	6.0	3090	4270	4910	2930	0	78
No 107	0.62	6.5	3050	4340	5410	3310	0	75
No. 55	0.62	6.4	2960	4280	5650	3510	1390	73
No. 103	0.62	6.0	3530	5480	6480	4030	1330	72
No 110	0.64	6.0	3660	5090	5560	2160	0	71
No 108	0.62	5.8	3690	4970	5550	2610	0	63
No 101	0.62	5.7	3800	5760	6690	3030	0	59
No. 89	0.62	5.6	4200	4890	6550	2400	0	50
No. 88	0.64	5.7	3570	4950	6660	2150	0	49
No. 63	0.60	5.8	2390	4610	5950	1900	0	49
No. 60	0.60	6.1	2300	3850	6380	1900^{a}	0	42
No. 33	0.60	5.7	3170	4880	5950	1730^{b}	0	36
No. 62	0.62	5.6	2480	4470	5760	1640^{c}	0	32
No 104	0.62	6.5	3030	4870	5160	2000^{d}	0	30
No. 111	0.64	5.9	3420	4960	6050	2210^{e}	0	29
No. 100	0.62	6.4	2860	4980	5740	1950^{f}	0	27
Average	0.62	6.0	3170	4810	5970	3650	1470	92

Tested when less than 1 year: ^{a} 45 weeks, ^{b} 39 weeks, ^{c} 35 weeks, ^{d} 33 weeks, ^{e} 32 weeks, ^{f} 30 weeks.
^{g} Estimated from length changes.

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[1] "Action of Alkali Salt on Portland Cement Mortars," *Engineering and Contracting*, Vol. LVII, No. 15, p. 359 (1922).

[2] "Volume Change a Measure of Alkali Action," *Public Roads*, Vol. 5, No. 4, p. 12 (1924).

TABLE III.—TESTS OF 2 BY 4-IN. CONCRETE CYLINDERS, 1:3 MIX, STORED IN MEDICINE LAKE, SOUTH DAKOTA.

These cylinders were made of cements from some of the same lots of portland cements that were used in the cylinders of Table II. Each result is average of 5 cylinders made on different days.

Cement	Water Ratio	Absorption at 21 Days per cent	Average of Compression Tests, lb. per sq. in.			
			Stored in Water			Medicine Lake
			7 days	28 days	1 year	1 year
No. 40	....	...	....	....	....	....
No. 97	0.64	6.4	3120	4440	5720	4600
No. 79	0.62	6.1	3220	4600	6170	4000
No. 41	....	...	....	....	....	....
No. 42	....	...	....	....	....	....
No. 34	0.66	6.1	3340	4540	5980	5090
No. 35	....	...	....	....	....	....
No. 113	0.64	6.3	2140	3940	6000	5590
No. 37	....	...	....	....	....	....
No. 98	0.64	6.1	4220	4970	7050	3400
No. 102	0.62	5.9	3860	5640	6630	4410
No. 105	0.62	5.7	3520	5390	6500	4300
No. 39	....	...	....	....	....	....
No. 106	0.62	6.1	3030	4410	6430	2490
No. 65	0.60	5.9	3150	3950	6050	2250
No. 107	0.62	6.5	2830	4540	5920	2940
No. 55	....	...	....	....	....	....
No. 103	0.62	6.0	3010	4380	5470	3160
No. 110	0.64	5.5	3660	4920	6910	4850
No. 108	0.62	5.8	3670	5010	6510	1490
No. 101	0.62	6.3	3680	4940	6290	2440
No. 89	0.62	5.9	4010	5220	6730	670
No. 88	0.64	6.1	3730	5040	6540	810
No. 63	0.59	5.8	2720	4290	5490	3190
No. 60	....	...	....	....	....	....
No. 33	0.59	5.4	3340	4780	5900	1590
No. 62	0.59	5.8	2960	3790	4580	800
No. 104	0.62	6.4	2940	4380	5530	280
No. 111	0.64	5.7	3810	5020	5860	3920
No 100	0.62	6.4	2900	4310	5110	0
Average	0.62	6.0	3310	4660	6060	2830

TABLE IV.—STRENGTH RATIOS, BASED ON ONE-YEAR TESTS OF 22 CEMENTS UNDER TWO DIFFERENT EXPOSURE CONDITIONS.

Cement	Per Cent of Normal Strength at 1 Year		Cement	Per Cent of Normal Strength at 1 Year	
	1-Per-Cent Sodium Sulfate	Medicine Lake		1-Per-Cent Sodium Sulfate	Medicine Lake
No. 97	98	80	No. 110	39	70
No. 79	86	57	No. 108	48	23
No. 34	88	85	No. 101	41	39
No. 113	87	93	No. 89	37	10
No. 98	83	48	No. 88	32	12
No. 102	81	67	No. 63	32	58
No. 105	70	66	No. 33	0	27
No. 106	64	39	No. 62	0	17
No. 65	60	37	No. 104	0	5
No. 107	61	50	No. 111	0	68
No. 103	62	58	No. 100	0	0

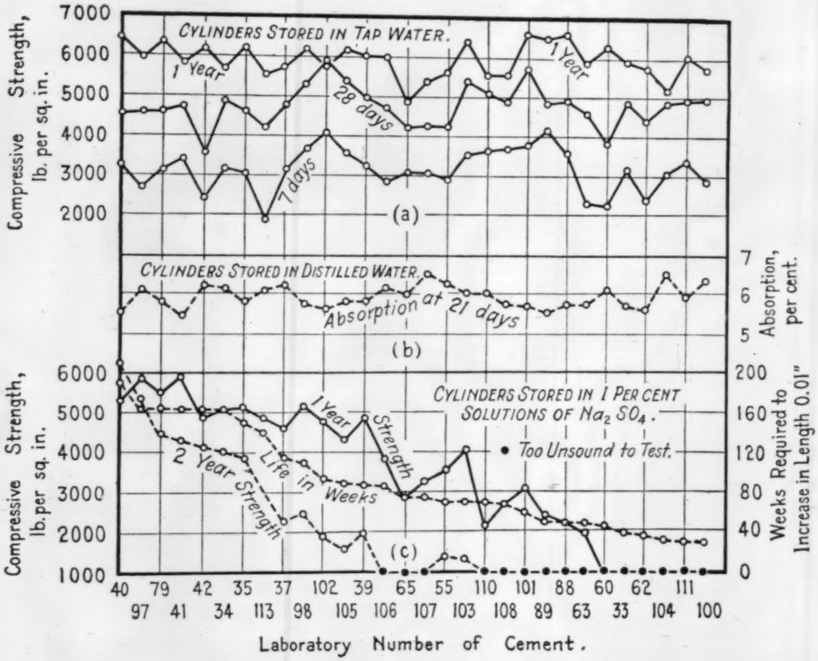
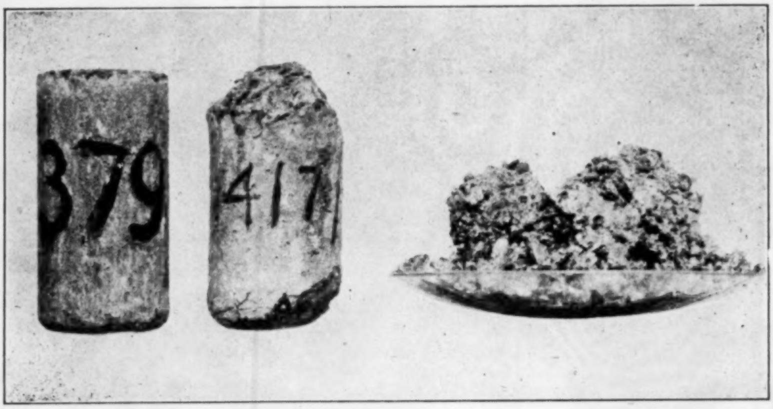


FIG. 1.—Standard Portland Cements from 30 Different Plants Compared to Show Influence on (*a*) Strength of Concrete, (*b*) on Absorption of Concrete, and (*c*) on Resistance of the Action of 1-per-cent Solutions of Sodium Sulfate (Na_2SO_4).



Cement 97 **Cement 106** **Cement 100**

FIG. 2.—Conditions at Two Years of Cylinders from Cements Nos. 97, 106 and 100 Varying Widely in Resistance. The cylinders of Nos. 97 and 106 fairly represent these series, while the cylinder No. 100 was better than average.

Results of tests of the cylinders stored in the laboratory solutions are compiled in Table II and shown graphically in Fig. 1. Figure 2 illustrates the wide range of conditions of cylinders of different series at 2 years. Results of tests of the Medicine Lake cylinders are given in Table III. The strength ratios of cylinders at 1 year, for both exposure conditions, have been calculated on the basis of the 1-year test of the check cylinders from the laboratory tank and these results are set forth in Table IV for the 22 cements subjected to the two different exposure conditions.

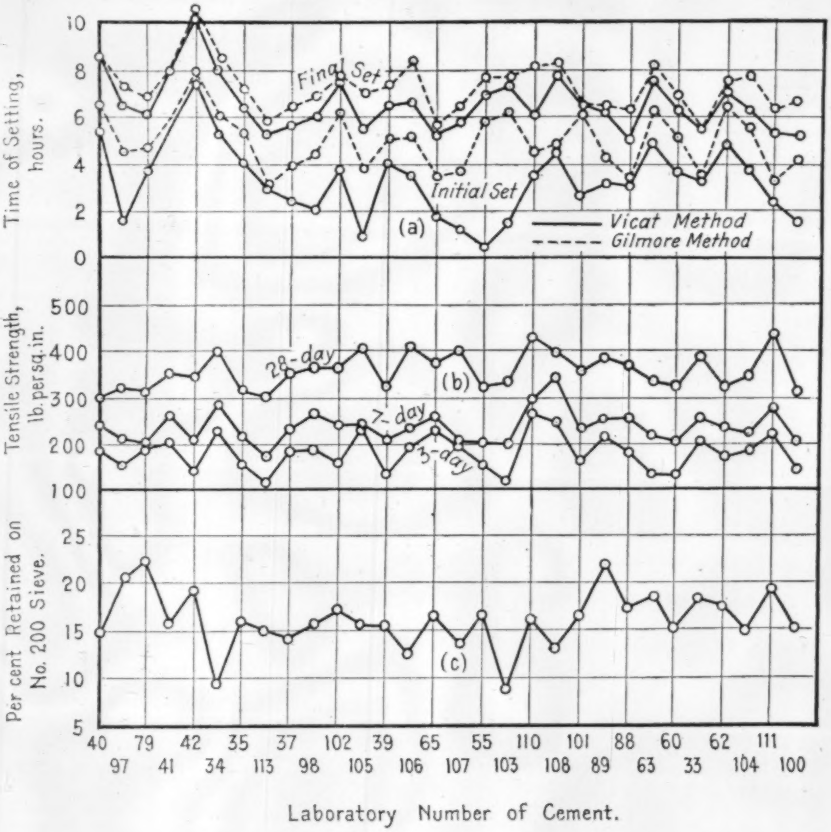


FIG. 3.—Standard Tests of 30 Cements for (*a*) Time of Set, (*b*) Tensile Strength, and (*c*) Fineness.

It is apparent from the data of Table IV that, while all of the 22 cements have not acted entirely consistently, the inconsistencies have been few and none of the cements making either an exceptionally

Table V.—Fineness of 7 Portland Cements Differing Widely in Resistance to Alkali, as Indicated by Results of Tests Recorded in Tables II, III and IV and Illustrated in Fig. 1.

Analyses other than No. 200 sieve in Table V were made by Bureau of Standards, U. S. Department of Commerce.[a]

Cement	Passing No. 200 Sieve, per cent	Passing No. 325 Sieve, per cent	Under 60 Microns, per cent	Under 40 Microns, per cent	Under 20 Microns, per cent	Under 10 Microns, per cent
No. 40	84.7	74.7	....	....	....	16.5
No. 41	84.1	74.6	66.2	52.1	34.0	15.9
No. 34	90.9	82.8	....	....	....	18.2
No. 55	83.8	75.3	65.3	51.9	34.3	17.0
No. 63	81.8	70.3	....	....	....	16.2
No. 33	82.1	71.9	....	....	....	15.9
No. 62	83.0	72.6	67.7	53.1	36.6	19.2

[a] A description of the apparatus may be found in the Bureau of Standards *Technologic Paper No. 48*.

Table VI.—Chemical Analyses of Portland Cements Used.

Chemical Analyses by the Division of Tests, Bureau of Public Roads, U. S. Department of Agriculture.
All values are given in per cent.

Cement	Silica (SiO_2)	Iron (Fe_2O_3)	Alumina (Al_2O_3)	Lime (CaO)	Magnesia (MgO)	Sulfuric Anhydride (SO_3)	Loss on Ignition	Total
No. 40	21.71	3.39	6.53	62.50	1.59	1.77	1.84	99.33
No. 97	21.45	2.65	6.45	62.50	2.97	1.82	1.95	99.79
No. 79	22.85	4.27	4.93	61.70	1.92	1.25	3.10	100.02
No. 41	21.13	2.73	8.89	62.40	0.49	1.78	1.29	98.71
No. 42	20.47	3.58	6.24	61.82	3.94	1.85	1.95	99.85
No. 34	22.43	3.20	5.03	62.54	3.75	1.91	1.20	100.06
No. 35	21.93	3.42	6.31	62.29	3.44	1.33	1.22	99.94
No. 113	24.87	2.39	4.57	61.30	1.26	1.58	3.60	99.57
No. 37	21.55	2.89	8.51	60.64	1.85	1.94	2.09	99.47
No. 98	22.90	2.82	5.88	63.70	0.94	1.72	1.95	99.91
No. 102	21.74	3.10	6.38	63.06	1.99	1.76	1.47	99.50
No. 105	22.45	2.33	5.92	62.70	1.79	1.87	2.70	99.76
No. 39	20.82	3.10	7.52	62.75	1.12	1.64	2.10	99.05
No. 106	22.40	3.22	6.83	62.65	1.34	1.22	2.15	99.81
No. 65	21.80	2.57	7.69	62.05	2.46	2.16	1.18	99.91
No. 107	21.50	2.80	6.30	62.08	1.21	1.80	3.70	99.39
No. 55	21.52	2.78	6.25	61.78	3.78	2.01	1.47	99.59
No. 103	21.52	3.30	5.63	61.40	5.00	1.34	1.53	99.72
No. 110	22.40	2.42	6.48	63.05	1.76	1.49	2.20	99.80
No. 108	21.15	2.70	7.40	63.35	1.77	1.37	2.25	99.99
No. 101	21.70	2.70	6.59	64.19	1.27	1.66	1.55	99.66
No. 89	21.60	3.46	5.04	62.75	2.68	1.68	2.75	99.96
No. 88	22.50	3.06	6.39	62.95	1.30	1.56	2.25	100.01
No. 63	22.66	2.51	8.12	61.87	1.05	1.72	1.60	99.53
No. 60	22.22	3.20	8.06	62.60	0.59	1.47	1.46	99.60
No. 33	21.12	2.48	6.82	61.24	5.24	2.01	1.90	100.81
No. 62	20.53	2.14	9.61	63.06	1.56	1.72	1.26	99.80
No. 104	20.73	2.85	6.23	61.16	4.95	1.61	2.06	99.59
No. 111	20.97	4.19	6.71	63.43	1.45	1.65	1.54	99.94
No. 100	21.28	3.14	7.72	61.33	2.65	1.53	1.90	99.55
Average	21.80	2.98	6.70	62.36	2.24	1.67	1.97	99.72

Table VII.—Chemical Analyses of Table VI Divided Into Three Groups and Averaged.

All values are per cent.

Group	Silica (SiO_2)	Iron (Fe_2O_3)	Alumina (Al_2O_3)	Lime (CaO)	Magnesia (MgO)	Sulfuric Anhydride (SO_3)	Loss on Ignition	Total
I	22.13	3.13	6.33	62.14	2.22	1.70	2.02	99.67
II	21.73	2.83	6.64	62.49	2.22	1.67	2.07	99.65
III	21.53	2.97	7.13	62.46	2.27	1.66	1.83	99.85

good or an exceptionally poor test under one condition showed a complete reversal under the other condition, cement No. 111 most nearly proving an exception.

PHYSICAL AND CHEMICAL TESTS OF THE CEMENTS

Results of the standard physical tests of the 30 cements are shown in Fig. 3 and it requires but cursory studies to see the futility of using any of the standard physical tests as an index of the behavior of any particular portland cement to the action of sulfate-bearing waters.

A special fineness test was made of 7 of the 30 cements, selected because of extreme showings made under exposure conditions, and the results of these tests are recorded in Table V. It is obvious from Fig. 3 and Table V that the difference in fineness, however considered, is so slight as not to account for a difference in the cements.

Chemical analyses of all 30 of the cements are recorded in Table VI. As is to be expected, there is some difference in the chemical composition, but authorities on the chemistry of portland cements are not in complete agreement as to the principal compounds formed by these various constituents and it is therefore not known that these chemical analyses can be so interpreted as to be of value.

The data of Table VI have been condensed and considered in Table VII on the basis of averages by dividing the 30 cements into three groups. Group I contains the first 10 cements of Table VI, Group II the second 10, and Group III the last 10. The average life of cylinders for the three groups (see Table II) has been respectively 155, 79 and 40 weeks, the life ratio for the groups closely approximating 4 to 2 to 1.

Examination of the data as averaged in Table VII fails to disclose any decided trend that appears to be of enough significance to account for the differences in resistance of the cements of the three groups. It is interesting to note, though, that the nearest approach to trend is in the Al_2O_3 column which, considered along with the Fe_2O_3 column, lends support to the Henri Le Chatelier[1] theory of long ago that cements low in alumina and high in iron are resistant to sea water. Unfortunately for this theory, reference to individual analyses of cements in the three groups (see Table VI) shows several outstanding inconsistencies for which no ready explanation is evident.

Lime-Silica Index.—It has been suggested by Merriman[2] that the "lime-silica index" should be considered in determining the quality of a cement. Following the method as outlined by Merriman, the "lime-silica index" has peen calculated for each of the cements and given in Table VIII, but it is not apparent to the author how these results, differing as they do so slightly, are of value in considering the relative resistance to the action of sulfate waters of these same cements, that show such small variations in essential physical and chemical characteristics as measured by the standard tests.

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[1] "Search for Materials Capable of Resisting Sea Water," *Annales des Mines*, May and June, 1887.

[2] Thaddeus Merriman, "Lime-Silica Index as Measure of Cement Quality," *Engineering News-Record*, Vol. 96, No. 16, p. 648 (1926).

RAW MATERIALS

With the exception of two Canadian brands, all the 30 cements came from American plants scattered throughout central and western United States and, consequently, the raw materials from which these cements were made cover a wide geological range. In attempting to account for differences this thought opens another field for speculation and after studying the data the impression remains that there does appear to have been some tendency for cements from plants closely located, or cements known to be made of similar raw materials, to behave alike. As examples, consider together the following cements: Nos. 33, 60 and 63; 34 and 39; 40, 97 and 98; and 62, 110 and 111. This is merely a thought and by itself has small value but, properly developed and considered, might in some way, not now foreseen, help explain basic differences that must exist in these cements that have acted so differently under identical exposure conditions.

TABLE VIII.—LIME-SILICA INDEXES OF 30 CEMENTS.

Cement	Lime-Silica Index	Cement	Lime-Silica Index	Cement	Lime-Silica Index
No. 40	2.16	No. 102	2.24	No. 101	2.31
No. 97	2.26	No. 105	2.22	No. 89	2.33
No. 79	2.14	No. 39	2.21	No. 88	2.17
No. 41	2.07	No. 106	2.14	No. 63	1.97
No. 42	2.27	No. 65	2.11	No. 60	2.02
No. 34	2.21	No. 107	2.24	No. 33	2.18
No. 35	2.16	No. 55	2.19	No. 62	2.14
No. 113	2.04	No. 103	2.25	No. 104	2.29
No. 37	1.91	No. 110	2.21	No. 111	2.27
No. 98	2.21	No. 108	2.27	No. 100	2.11

BRIQUET TESTS FOR SULFATE RESISTANCE

Irrespective of the fundamental causes for the difference of behavior of these 30 cements, there most certainly has been a decided and consistent difference. Granted that the only possibility for the ultimate solution of this whole problem is by the chemist, it is clear that the immediate necessity for engineers is a satisfactory standard test for cements to be used in concrete to be exposed to the action of sulfate-bearing waters, that very poor cements may be eliminated from consideration. With this in mind a series of experiments was conducted looking towards an accelerated test, with results in most cases far from satisfactory. It was found, however, that when the standard briquets used in the 7-day test were immediately stored in 1-per-cent solutions of sodium sulfate their behavior subsequently, judged by appearances, seemed to agree surprisingly well with the tests of the cylinders. The 7-day briquets of 16 of these same 30 cements were used in these briquet tests, so there is a complete record of 16 cements under the four distinct exposure conditions:

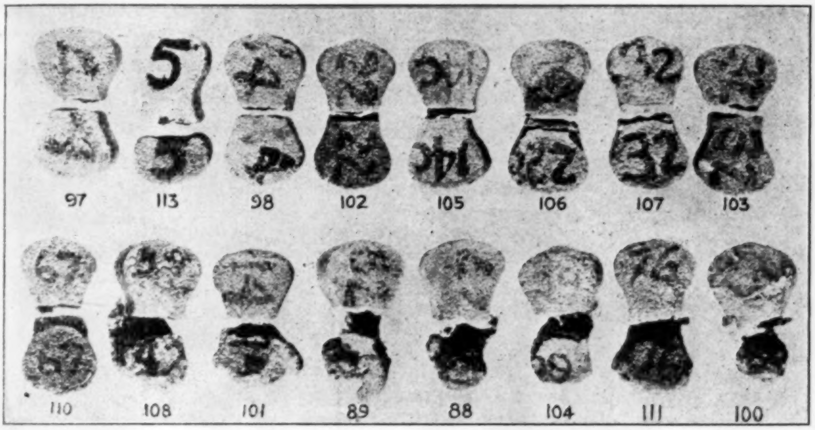

FIG. 4.—Views of Standard Briquets After 6 Months' Exposure in the Laboratory to 1-per-cent Sulfate Solutions. Upper halves were in magnesium sulfate ($MgSO_4$) while lower halves were in sodium sulfate (Na_2SO_4). Standard portland cement from a different plant was used in each of these 16 briquets.

1. In cylinders without any air hardening exposed in the laboratory to 1-per-cent solution of sodium sulfate;
2. In cylinders after 5 weeks in air exposed to the mixed salts of Medicine Lake;
3. In half of a standard 7-day briquet exposed in the laboratory to 1-per-cent solution of sodium sulfate; and
4. In half of a standard 7-day briquet exposed in the laboratory to 1-per-cent solution of magnesium sulfate.

In Fig. 4 are photographs of the briquets from the laboratory solutions at 6 months, in all cases the upper half coming from 1-per-cent magnesium-sulfate solutions and the lower half from 1-per-cent sodium-sulfate solutions. It is evident from this figure that action on the half briquets in the sodium-sulfate solution had progressed much further at 6 months than had action in the magnesium-sulfate solution. Considered alone from the standpoint of an accelerated test the half briquets from the 1-per-cent sodium sulfate solution are the most promising.

In making these briquet tests the quantity of each solution was at no time less than 1 gallon for each 15 briquets and the interval to solution change in no case exceeded 4 weeks. All solutions were kept at room temperatures of about 72° F.

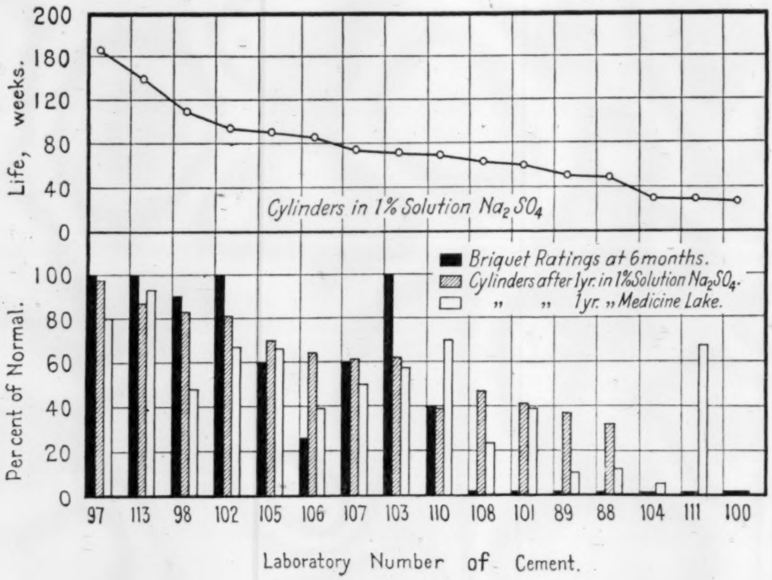

FIG. 5.—Resistance to the Action of Sulfate Waters of Standard Portland Cements from 16 Different Plants as Determined by Different Conditions of Exposure. Note the general reliability of the briquet ratings by visual appearances as compared with the change of volume and compression tests of the cylinders.

In the lower half of Fig. 5 the ratings at 6 months on the basis of visual appearances of the half briquets from the 1-per-cent solutions of sodium sulfate (see Fig. 4) are compared with the strength ratios at one year of cylinders in which were used the same cements (see Table IV). Study of Fig. 5 justifies the statement that the acceptance or rejection of any of these 16 cements on the basis of the appearance of the briquets at 6 months would not have been very far wrong, when the action of the cylinders under both exposure conditions is considered. In connection with Fig. 5 it is to be remembered that the bases for comparison are tests of specimens made entirely independently of each other without, at the time of making, having in mind correlating the data as has been done. Furthermore, these are in no sense selected tests as the only reason for choosing the particular 16 cements for Fig. 5 is that they were the only portland cements on which the three-way test had been made.

It would seem from the behavior of the 7-day briquets of these 16 cements that a very satisfactory 6-months test for resistance to the action of sulfate waters can be made by following substantially the same routine, testing several cements simultaneously in 1-per-cent solutions of both sodium sulfate and magnesium sulfate, and rejecting the cements used in any briquets giving evidence of more than very slight surface action after 6 months in either solution.

The feasibility of speeding up this 6-months test by increasing the strength of the solution, and by using leaner mixes naturally suggests itself. Attempts along this line have been made in the laboratory, and are still under way, without, as yet, enough satisfactorily consistent results to justify discussion except to say that the possibilities along this line appear good, although too much can not be expected. Therefore, as a step in the right direction and until a more accelerated test of at least equal reliability is developed, the one outlined is suggested.

The 6-months briquet test has the three distinct advantages of (1) reliability, (2) close approximation of severe, but not extreme, field conditions, and (3) not requiring the making up of additional and special test pieces. It has the disadvantage of requiring 5 to 6 months for completion.

If it may be assumed that the resistance of a cement from any plant is reasonably constant, the time element is not serious as tests need be made only often enough to make certain that no plant change has brought about a change of resistance. On the other hand, if this assumption is incorrect and the output of a cement plant varies greatly in resistance, the time required for the test becomes a serious objection. The evidence in the laboratory, along this line, seems to indicate that the resistance factor of a cement from any given plant is fairly constant. Complete and exact data, however, to support this statement are at this time lacking and a series of tests with 8 brands of cements, definitely planned to clear up this point, is already well advanced.

SUMMARY AND CONCLUSIONS

1. Standard portland cements from different manufacturing plants may vary greatly in resistance to the action of sulfate waters, as evidenced by laboratory and field tests of 30 portland cements after

exposure periods ranging upwards to more than 3 years. Under the same exposure conditions the more resistant cements have outlived those of least resistance by as much as eight times, while the most resistant 10 have had an average life very nearly four times that of the 10 of least resistance.

2. Portland cements that have failed quickly in the laboratory in pure solutions of sodium sulfate have ordinarily displayed low resistance in the field to the action of mixed salts.

3. The most desirable portland cements for concrete exposed to the action of sulfate waters are those that prove most resistant to the action both of pure salts and of mixed salts.

4. Results of standard physical tests of portland cements give no indication of resistance to sulfate waters.

5. Results of ordinary chemical analyses of portland cements give no indication of resistance to sulfate waters.

6. The geological differences of the raw materials of different cements may possibly be one factor that must be considered in attempting to account for differences in resistance to sulfate waters.

7. A satisfactory accelerated test is needed to facilitate determining the resistance of a portland cement to the action of sulfate waters. Until a better test is developed the 6-months briquet test, as outlined in this paper, is suggested.

With so great a difference in resistance to the action of sulfate bearing waters, most certainly the first consideration for all concrete to be exposed to such action should be the cement itself and regardless of all other precautions the use of any cement of low resistance, as determined by special tests, should be avoided.

DISCUSSION

MR. JOHN R. BAYLIS[1] (*presented in written form*).—This excellent paper by Mr. Miller gives another illustration of how inadequate are existing methods for testing portland-cement concrete. Efforts to add to existing tests, or to bring out new tests, probably will be resisted by many who think only in terms of early-strength tests. A few now realize that existing methods for testing cement and concrete fall short of our needs for concrete under certain conditions of exposure. **Mr. Baylis.**

Mr. Miller's attempt to find some means by which the resistance of concrete to sulfate waters may be determined is to be commended. Additional work may show this to be far short of our needs, but it is a start on something that may lead to a better understanding of why concrete under similar conditions of exposure may vary greatly in its resistance to the elements.

The part that the sulfate radical (SO_3) plays in the setting of portland cement and the part it plays in its durability has been of great interest to the writer. It is believed that when all is known about the chemical reactions of calcium, aluminum and sulfates a great advance will have been made in our knowledge of the characteristics of portland cement.

The durability of concrete exposed to sulfate waters is largely one of how quickly the SO_3 can get to the cement compounds. If all of the calcium aluminate was exposed the reaction could be measured in hours rather than in weeks. The thought which suggests itself, basing it on the evidence submitted by Mr. Miller, is that some brands of cement are more effective in precipitating protective layers over the particles of cement containing calcium aluminate than are other brands. It may be that some of the minor constituents of the cement, such as sodium and potassium, also play an important part.

Several years ago the writer made a number of tests on the soluble constituents of portland cement in its early stages of setting. It was found that the SO_3 in solution varied quite materially for several brands of cement, and the sodium and potassium content of the cement appeared to be the major factor influencing the variation. All of the SO_3 in portland cement soon goes into combination with the cement compounds and the solution within concrete 12 to 24

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[1] Physical Chemist, City of Chicago, Chicago, Ill.

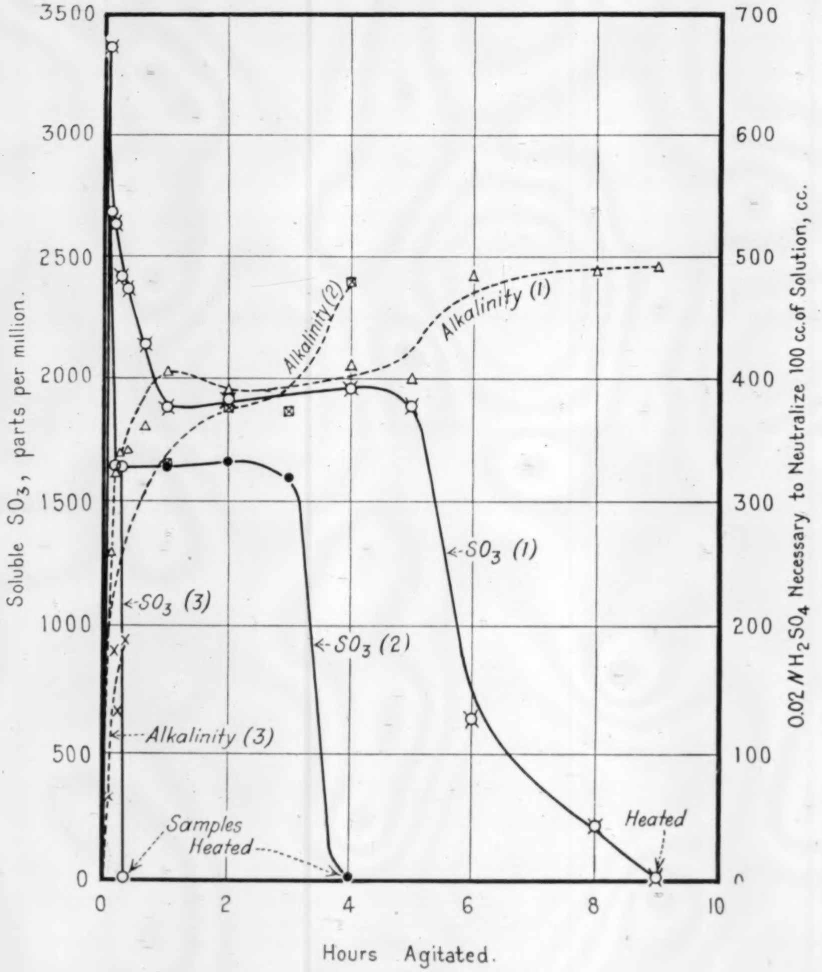

FIG. 1.—Soluble Sulfates and Alkalinity of a Solution Containing Atlas Portland Cement.

(1) Fresh cement purchased from a dealer in Baltimore. Reported to have been in the warehouse less than one month.

(2) Sample from the center of a bag stored in the laboratory for 2 years.

(3) Sample $\frac{1}{2}$ to $1\frac{1}{2}$ in. from the outside of the bag. Fairly hard, but easily pulverized with the hand.

200-g. samples were added to 225 cc. of distilled water and the sample agitated continuously in a rotating machine. Separate samples were used for each test.

Mr. Baylis.

hours after the water is added is practically free from SO_3. The kind of crystals formed by the union of SO_3 with calcium aluminate is not known. They appear to be very small and certainly are not the characteristic needle crystals of calcium sulfoaluminate found so abundantly in concrete exposed to water. The needle crystals represent a change taking place in the concrete after it has set. The writer has heard the opinion expressed that the needle crystals are actually formed in the setting of the cement, but are too small to be detected. The evidence seems to be against this opinion (see the accompanying Fig. 4).

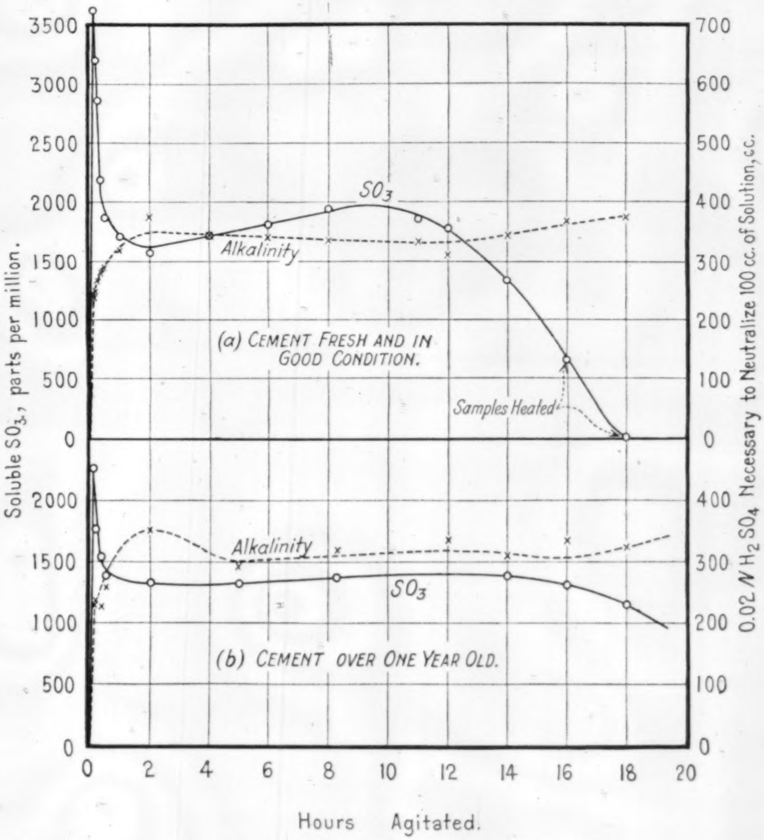

FIG. 2.—Soluble Sulfates and Alkalinity of a Solution Containing Cement.

200-g. samples of portland cement were added to a flask containing 225 cc. of distilled water, and agitated continuously in a rotating machine. Separate samples used for each test.

In Fig. 2 (*b*) the bag of cement was stored in the cellar of a residence about 14 months. The sample was collected from near the center of the bag.

Mr. Baylis. Samples of cement before hydration which have been stored a year or more were available for two of the brands of cement. One brand showed that the time required for the SO_3 to go into combination with the cement compounds was materially lengthened after being stored, whereas the other cement showed it to be materially shortened. This is shown in the accompanying Figs. 1 and 2. No

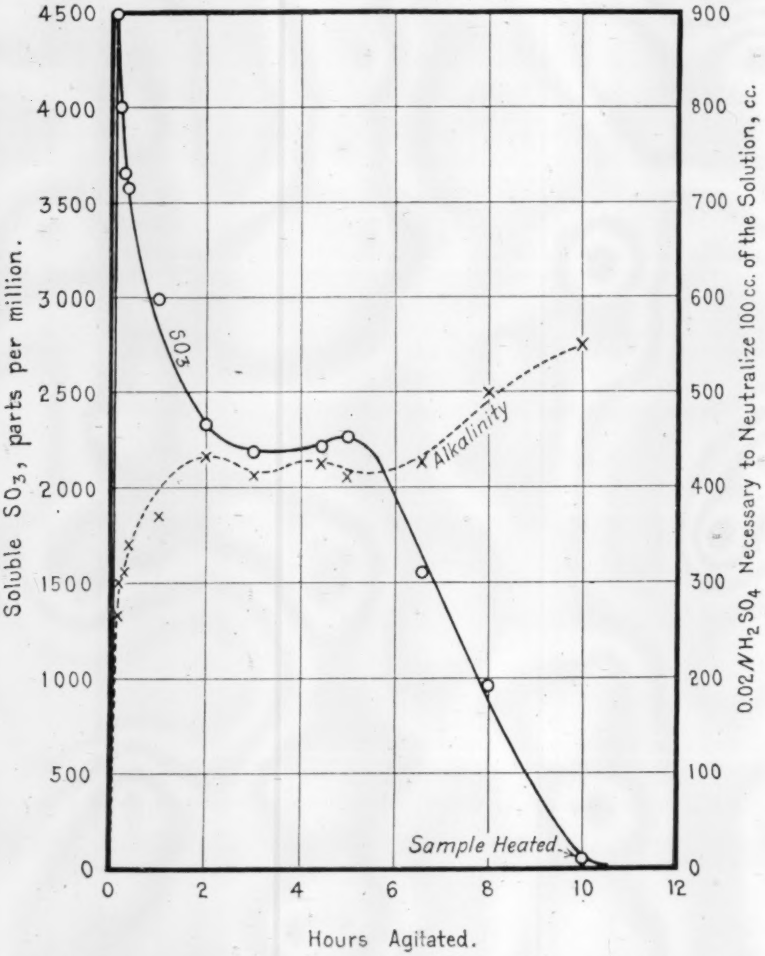

FIG. 3.—Soluble Sulfates and Alkalinity of a Solution Containing Alpha Portland Cement.

200-g. samples were added to a flask containing 225 cc. of distilled water, and agitated continuously in a rotating machine. Separate 200-g. samples were used for each test. The cement was purchased from a dealer in Baltimore and appeared to be in excellent condition. It was reported to have been in the warehouse about two weeks.

Mr. Baylis.

reason as to why this should be the case can be offered. The fact that each point on the curve represents a separate portion of the sample prevents an error of any material consequence being made. Contrast the SO_3 curves for the fresh cement in Figs. 1 and 2 with the SO_3 curve in Fig. 3, which is a cement high in sodium and potassium.

These illustrations are given with the idea of showing that there are marked differences in the soluble constituents of several brands of cement. This may not have any bearing whatever on the facts brought out by Mr. Miller, but it does show differences not readily explainable. With a product of such great importance as portland cement and one of which so little is known it seems that any information aimed at a better understanding of its characteristics should be made public. It is such papers as that presented by Mr. Miller,

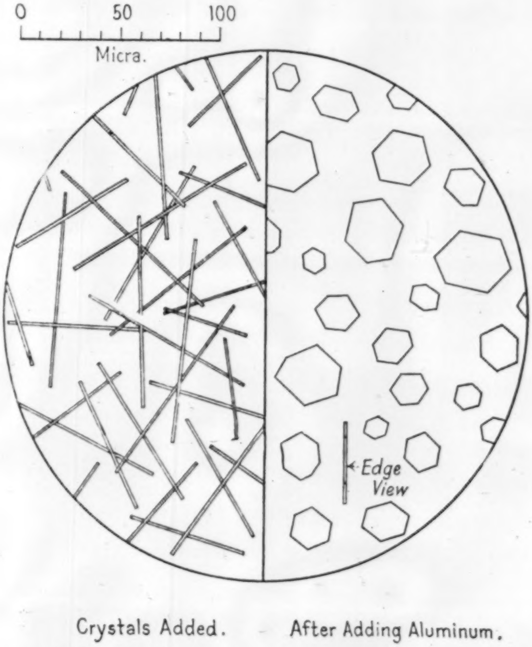

FIG. 4.—Solution Containing Hydrating Calcium Aluminate Dissolves Calcium Sulfoaluminate.

A large number of calcium sulfoaluminate crystals were added to a solution containing a large excess of calcium hydroxide. Aluminum metal was then added. There was considerable liberation of hydrogen gas and after 24 hours the camera lucida drawing shows that all needles had gone into solution. Only flat hexagonal crystals remained. It is very likely that a certain amount of SO_3 enters into the hexagonal crystals, and that the continued production of the crystals will reduce the concentration below the equilibrium necessary to prevent the needle crystals from dissolving.

Mr. Baylis. giving experimental facts even though no reason for the results can be offered, that will arouse interest among those capable of solving such problems.

Mr. Williams. MR. G. M. WILLIAMS[1] (*presented in written form*).—The great variation in the endurance of different brands of portland cements exposed to sulfate solutions as reported by Mr. Miller is of especial interest to the writer in view of the results obtained in a similar study conducted under the auspices of the Committee on Deterioration of Concrete in Alkali Soils of the Engineering Institute of Canada. This latter investigation, devoted mainly to chemical research and supplemented by field exposure tests of small groups of concrete specimens in each of the three Prairie Provinces, has been under way since 1921, and a report of field tests to date together with a discussion of the relative resistance of the products of different mills was presented at the annual general professional meeting of the Engineering Institute at Montreal in February, 1928. The report appears in full in *The Engineering Journal*, March, 1928.

The discussion on the relative resistance of various cements to the action of sulfate waters, furnished by Mr. T. Thorvaldson who has directed the chemical laboratory research of the committee, points out the marked differences found for a number of Canadian brands of cement studied. Earlier work showed the relation between expansion of a test specimen in the form of a mortar bar $\frac{5}{8}$ by $\frac{5}{8}$ by $7\frac{1}{2}$ in. and decrease in tensile strength of the same mortar when molded and tested in the form of a briquet. The use of lean mortar mixtures by the expansion bar method permits of the determination of the relative resistance of different cements in from 20 to 30 days. While it is not possible to compare directly the results with those reported by Mr. Miller owing to the use of different cements, mixtures, and other conditions of test, the two laboratories appear to be in close agreement in pointing out that the output of different mills differs widely in sulfate resistance, and also that these resistances can be accurately measured in the laboratory by strength and expansion tests.

Both laboratories also point out that the usual routine physical tests of a cement, as well as the compressive strength of a mortar or concrete, in no way serve as a measure of resistance to sulfate action. It also appears that the usual chemical analysis of a cement offers little information as to sulfate resistance although as Mr. Miller indicates there seems to be a general trend towards increased life as the percentage of alumina decreases. A recent report by Thorvaldson

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[1] Professor of Civil Engineering, University of Saskatchewan, Saskatoon, Sask., Canada.

Mr. Williams.

and associates[1] outlining the results of tests with the components supposed to be present in normal portland cement when exposed to sulfate solutions states that (1) mortars of pure tricalcium silicate and pure beta-dicalcium silicate are not disintegrated in solutions of sodium sulfate but expand and are disintegrated in solutions of magnesium sulfate, (2) that mortars made from tricalcium silicate or dicalcium silicate to which tricalcium aluminate has been added disintegrate rapidly in solutions of both sodium and magnesium sulfate, (3) that a 1 : 7½ mortar made from a composite cement containing 21.5 per cent of tricalcium aluminate, 52.7 per cent of tricalcium silicate and 25.8 per cent of beta-dicalcium silicate has the usual strength of a similar mortar made from a normal portland cement, and (4) mortar bars made from this composite cement behave like mortars from normal portland cement during disintegrations in solutions of sodium sulfate, magnesium sulfate, and calcium sulfate. Incidentally both Miller and Thorvaldson have found that steam treatment of specimens at temperatures above the boiling point of water results in greatly increased resistance. Thorvaldson states[2] that this treatment causes an enormous increase in the resistance of a mortar to the action of solutions of sodium sulfate and that while this same treatment does not render mortars immune to the action of solutions of magnesium sulfate the action is very much retarded.

As stated by Mr. Miller, a simple accelerated test which will permit of quick and accurate determination of the sulfate resistance of a cement is especially needed. Since it may be found that variation in control of mixture and burning in any one mill may result in appreciable variation in the quality of the output of that mill from day to day, it may prove desirable to make the resistance test on every shipment or smaller unit of quantity, so that the time element in any test adopted may be an important factor. For this reason the writer would prefer an accelerated expansion test on lean mortar mixtures, but even such a test will require days instead of hours which would seem most desirable. The writer has always looked favorably upon some such high pressure steam test for cement as was proposed by Mr. H. J. Force fifteen years ago.[3] In the preparation of cement specifications there has been a tendency to ignore this type of test, no doubt due to lack of knowledge as to what changes take place in the cement, but as more information is obtained with regard to the

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[1] Thorvaldson, Vigfusson and Larmour, "The Action of Sulfates on the Components of Portland Cement," *Transactions*, Royal Soc. Canada (1927).

[2] Thorvaldson and Vigfusson, "The Effect of Steam Treatment of Portland Cement Mortars on Their Resistance to Sulfate Action," *The Engineering Journal*, Vol. XI, No. 3, p. 174.

[3] H. J. Force, "Results Obtained with the Autoclave Tests for Cement," *Proceedings*, Am. Soc. Testing Mats., Vol. XIII, p. 740 (1913).

Mr. Williams. constitution of cement and the reactions which occur during the process of hydrating and hardening over long periods of time it is possible that such a test may be found of value as a substitute for one or more of the present physical tests. The little work which the writer has carried out with cements differing in sulfate resistance when exposed in the form of mortars to high-pressure steam indicates that it may be possible to select some pressure and period of exposure which will evaluate the sulfate resistance of a cement in 24 hours or less.

Where practicable to steam cure a concrete it is possible to use portland cement as now constituted with assurance of long life in sodium sulfate waters and studies now under way give promise of furnishing an easier means of greatly prolonging life in magnesium sulfate solutions. To the user of concrete in mass form it probably seems that little progress has been made in reaching a satisfactory solution of the alkali-concrete problem, but the studies of cement and its constituents now under way in several laboratories will no doubt in time attain the desired end. In the meantime, as Mr. Miller points out, more attention to the selection of the particular cement to be used in concrete which may come in contact with sulfate solutions may result in practically a normal life for a structure which if made with a less-resistant cement might require early replacement or repair.

Mr. Thorvaldson. MR. T. THORVALDSON[1] (*presented in written form*).—Mr. Miller's conclusion that the "standard physical tests of portland cements give no indication of resistance to sulfate waters" is in agreement with the results of laboratory experiments carried out by the writer at the University of Saskatchewan and published in part as Appendix A to the 1927 Report of the Committee on the Deterioration of Concrete in Alkali Soils of the Engineering Institute of Canada.[2] About thirty portland cements from nine different mills were used in these experiments, the comparison being made by determining the expansion[3] of lean mortar bars in solutions of sodium and magnesium sulfate of several different concentrations. No relation was found to exist between the tensile or compressive strengths developed by the cement in 1:3 standard sand test specimens and its resistance to sulfate action. Of the cements tested the one giving the highest tensile and compressive strength was found to be the least resistant to sulfate action while the one of lowest strength was the next lowest in sulfate resistance. The cements tested showed great variation in sulfate

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[1] University of Saskatchewan, Saskatoon, Sask., Canada.

[2] *The Engineering Journal*, March, 1928.

[3] Thorvaldson, Larmour and Vigfusson, "Expansion of Portland Cement Mortar Bars During Disintegration in Sulfate Solutions," *The Engineering Journal*, Vol. 10, p. 199 (1927).

Mr. Thorvaldson.

resistance, some possessing more than five times the resistance of the two cements mentioned above when exposed to a 6-per-cent solution of sodium sulfate. While the comparisons were generally made with lean standard sand mortars (1:5, 1:7½ and 1:10) similar experiments with several cements using richer mixes and a well-graded sand gave essentially the same relative values for the sulfate resistance of the cements. It seems, therefore, that an accelerated test for comparing the sulfate resistance of portland cements by determining the rate of expansion of lean mortar bars in sulfate solutions is practicable.

Mr. Draffin.

MR. JASPER O. DRAFFIN[1] (*by letter*).—This is an interesting and a valuable paper, which I infer is more of a progress report than the report of a finished investigation. Some of the results might have been anticipated, but of course a confirmation of them is necessary. The proposed test for durability should be valuable if the indicated uniformity of cement from the different mills is true. The author is to be commended for the completeness of the data which he presents.

The data given in the paper show such a wide variation in the durability of the different cements that the question of composition comes to mind immediately. There must certainly be a difference in the composition of the different cements even if that difference is not apparent from the chemical analyses. When two cements of almost identical composition, as for instance cements Nos. 41 and 60, differ so greatly in their resistance to sulfate waters there can be but one conclusion concerning their composition and that is that the real composition is not completely determined by the conventional chemical analysis. The statement that geological differences in the origin of the raw material may be a factor is important in this respect. It suggests that chemical composition is not a sure criterion of the mineral constituents present in the finished cement and this is the essential point. Any prediction as to strength or durability presupposes a certain type of equilibrium and chemical action in the cement during the burning, even if its character is unknown. If this equilibrium is changed in any way the expected results will probably not be attained. If the difference in the raw materials is a factor it must be so because different temperature conditions are needed to produce the same normal equilibrium or else because some minor constituents which are usually neglected in the analysis have a powerful effect, as catalyzing or retarding agents, in modifying the equilibrium. A fundamental study of the compounds actually present in different raw materials and in portland cement manufactured under different conditions is necessary before chemical composition and durability or

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[1] Assistant Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

Mr. Draffin. physical strength can be completely correlated. It cannot be emphasized too strongly that while we know the compounds which should be present in a mixture of pure Ca-Al-Si brought to a state of equilibrium, we do not in practice deal with pure materials and therefore any variation from the pure materials will produce a corresponding, but at present unknown, effect on the compounds in the cement.

Since the author has referred to the "lime-silica index" and other means of judging of the suitability of a cement, attention is called to a paper by Messrs. Atwood and Johnson[1] which contains a large amount of valuable information gathered from many sources. As might be expected, there is more or less contradiction and discrepancy between the results reported by different experimenters working under different conditions. But Messrs. Atwood and Johnson summarize the different results by saying, ". . . the primary cause for the disintegration of mortar and concrete in sulfate-carrying waters, such as sea water and many alkali waters, is the attack on the free lime in the mortar by the sulfates in the water. The majority . . . agree that this disintegration can be prevented by the addition to standard portland cement of a properly constituted siliceous material. . . . The high-alumina cements attain the same results by different means. . . . The use of a single standard specification for the binding agent . . . does not seem to be desirable or efficient."

Mr. Miller. MR. D. G. MILLER[2] (*author's closure by letter*).—Mr. Baylis has found that the influence of long-time storage on the soluble constituents of cement is markedly different for different brands. This is one possible explanation for some of the apparent inconsistencies generally found in reports dealing with the action of sulfate waters on concrete. In the Minnesota experiments comparative tests have been made on certain fresh cements shipped direct from the mills, on the same cements after several months' storage, and on other lots of the same brands obtained from local warehouses. To date these data have been analyzed only superficially but it does not appear that, in general, the duration of time of storage has had much influence on the resistance of those cements observed. It is, however, entirely possible that more detailed studies of the data may reveal certain exceptions of the nature reported by Mr. Bayliss in his solubility tests.

The suggestion of Mr. Williams that a high-pressure steam test might be developed to speed up the determination of the sulfate resistance of a cement is certainly worthy of thoughtful considera-

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[1] *Transactions*, Am. Soc. Civil Engrs., Vol. LXXXVII, p. 204 (1924).

[2] Drainage Engineer, Bureau of Public Roads, U. S. Department of Agriculture, St. Paul, Minn.

Mr. Miller.

tion in view of the fact that the curing of concrete in steam at the boiling point of water, and superheated, raises to a remarkable degree resistance to the action of the sulfates of sodium and magnesium. With such evidence to show the extreme sensitiveness of portland cement to heat treatment during the critical period of hardening, it certainly appears well worth while to consider the use of steam as an agency in the search for an accelerated test for sulfate resistance.

The published results of Mr. Thorvaldson's experiments at the University of Saskatchewan have agreed in all essentials noted with the conclusions based on the Minnesota work if a single exception be made. Extremely lean mortar mixes for the high-alumina cements have not, in some cases, shown exact agreement with results obtained using more nearly normal mixes. This exception, in the case of a material greatly differing from standard portland cement, is perhaps of small moment in considering lean mixes as the basis for an accelerated test for standard portland cement.

Mr. Draffin's well stated argument for more fundamental studies of the composition of the different raw materials used in portland cement manufacture, and also of portland cement manufactured under different conditions, is of course the ultimate answer to this whole problem. This does not, however, minimize the immediate necessity among engineers for a special standard test and Mr. Draffin's citation from the comprehensive paper by Messrs. Atwood and Johnson supports this view.

The addition of siliceous materials to portland cement is mentioned in the Atwood and Johnson reference. This subject cannot be discussed here any more than to say that the writer is inclined to the opinion that undue optimism along this line is not entirely justified for, although many European investigators favor such additions, American investigators have been much less in agreement as to their value.

SOME ACCELERATED FREEZING AND THAWING TESTS ON CONCRETE

By C. H. Scholer[1]

Synopsis

This paper presents a method for testing the durability of concrete by alternate freezing and thawing at low temperatures. Data are included covering the field investigation of disintegrating concrete structures which show the relation between field conditions and the results of the laboratory investigation by the accelerated freezing tests. The data include results on unsound aggregates, varying values of water-cement ratio, and variations in rate of freezing under different conditions of exposure.

Introduction

The method of testing the durability of concrete by alternate freezing and thawing as presented in this paper is the outgrowth of about four years of investigation of concrete disintegration in Kansas highway structures. The work has been carried on by the Engineering Experiment Station of the Kansas State College under the direction of the author. The disintegration was occurring in the abutments and piers of structures on the streams of the Arkansas and Cimarron river valleys particularly, and as the water in most of these streams was slightly alkaline, the failure was naturally attributed to alkali action.

A careful survey of the structures in question revealed the following facts:

All failures were occurring in rather lean concrete mixtures, 1:3:5 or leaner.

In many of the structures showing disintegration the coarse aggregate was a limestone from ledges now known to be unsound, and an examination of the concrete showed the stone to be disintegrating and contributing largely, if not wholly, to the action. (See Fig. 1.)

Due to the scarcity of coarse aggregate in this section of the state, the use of mortar concrete using a coarse sand carrying some gravel is common. Much of this concrete was too lean, in some cases as lean as 1:9. Some of the older structures using this material built under state supervision with proportions of 1:4 about 1913 were in perfect condition, and later additions to the same structure were badly disintegrated. (See Fig. 2.)

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[1] Professor of Applied Mechanics, and Engineer of Tests, Kansas State Agricultural College, Road Materials Laboratory, Manhattan, Kans.

In general, the concentration of salts, especially the sulfates in these streams, was so low as to be unlikely to cause serious trouble. (See Table I.)

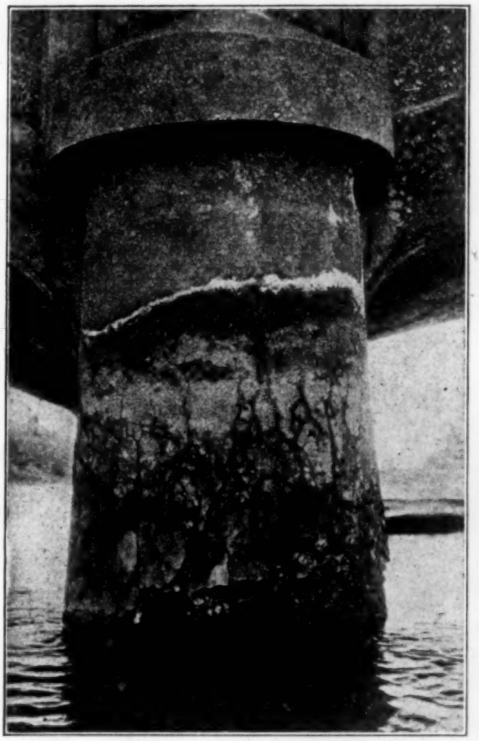

FIG. 1.—Pier of Main Street Bridge over Arkansas River, Hutchinson, Kan.

This pier was built in 1905 using the coarse aggregate of Series A. Proportion of mix, 1:3:5. See Fig. 5 for results of freezing and thawing tests using this aggregate.

Some of the worst cases were in the purer water and where the highest concentration of salts was found no action was indicated.

A study of structures in other sections of the state where the same or similar aggregates and lean mixes were used, and where the stream water was strictly fresh, revealed the fact that exactly the same type of disintegration was occurring.

After reviewing these facts, it was concluded that the real cause of the disintegration was the general lack of durability in the concrete itself rather than the severity of the conditions to which it was exposed; and that we were concerned not so much with a method of protecting our concrete from certain unusual and severe conditions, but in generally improving the quality from the standpoint of durability to resist ordinary weathering conditions. A search of technical literature for data as to the relative durability of concrete of varying quality revealed practically nothing. Investigators have, until recently, done little but make very general statements about this very essential quality of concrete, the general idea apparently being that all concrete possessed the property to a very marked and satisfactory degree.

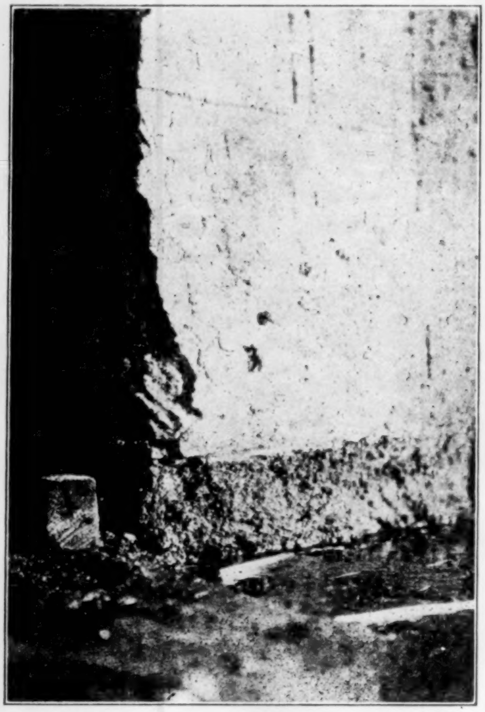

FIG. 2.—Bear Creek Bridge in Clark County, Kansas.

Showing character of disintegration of mortar concrete. The original structure at this site was built under State supervision about 1913. Proportion of mix, 1:4. The concrete in the original structures is in excellent condition. Disintegration is occurring in extensions made several years later. No record is available as to the mix used in the later work.

The investigation, as planned, consisted of two lines of study; one in the field, and the other in the laboratory. Only enough of the details of the field investigation will be given in this paper to connect the laboratory investigation with field conditions.

FIELD INVESTIGATION

The field investigation was planned to include a careful survey of existing structures and their condition, and to ascertain where possible the character of the aggregates used in the concrete as well as the quality of concrete itself, and the type of exposure at each site. In making this survey the following form was used:

DURABILITY OF CONCRETE IN ALKALI AND SULFATE WATERS

Field Data on Structures:

Name of Structure..

Type of Structure..

Name of Stream..

Location (Sec. twp. range)..

Character of Concrete Exposed to Water:

Age (give date poured if possible)..

Aggregates Used:

Sand..

Stone or gravel..

Cement, brand if possible..

Tests of each if made..

Approx. proportion of mix..

Consistency of mix..

Evidence of disintegration..

Alkali Stain..

Scaling..

Bulging or sloughing off and to what extent..

....

Location of above with reference to water line..

Proximity of industrial plants..

Characteristics of flow in stream and conditions at point of disintegration..

....

Are tests specimens planted here?..

Inspected by..

Accompanied by..

On reverse side make sketch of structure and location showing point where disintegration is most in evidence. Substantiate by photographs if possible. Secure sample of water for chemical analysis.

To check our conclusions as to the character of the action observed in the field, it was decided to plant field specimens of concrete of varying known quality and to observe the action of the elements on these specimens. A sufficiently wide range was planned to give concrete so poor in quality as to insure early results. These have been secured.

Field specimens of concrete were planted in locations as follows: at Hutchinson in the Arkansas River; at Lakin in the Arkansas River (see Figs. 3 and 4); at Ashland in Bear Creek; and at Englewood in Big Sandy Creek. At all of these locations except Lakin, extensive disintegration of concrete was occurring. At Lakin a concrete structure built in 1916 of the local sand and gravel aggregate is still in excellent condition. It is interesting to note that the most rapid disintegration of field specimens is occurring at this place.

TABLE I.—CHEMICAL ANALYSES OF WATER FROM STREAMS OF SOUTHWESTERN KANSAS. SAMPLED ABOUT JULY 1, 1926.

All Values are Parts per Million

	Ca	Mg	Cl	SO_4	CO_2 Combined	NaCl, Equivalent of Cl	Na_2SO_4 Equivalent of SO_4	Total Solids
Arkansas River (Oxford)	54	10	256	94	68	422	139	713
Argonia (Chica-Kia)	52	11	14	48	89	23	71	291
Arkansas River (Hutchinson Main St. Bridge)	178	88	4 620	517	31	7 617	765	8 661
Cow Creek (Hutchinson Pioneer St. Bridge, built 1916)	94	18	462	143	99	762	212	943
Arkansas River (Ellinwood)	153	35	84	537	64	138	794	1 160
Cimarron River (Freedom, Okla.)	300	139	18 200	807	47	30 005	1 194	31 496
Bluff Creek (North of Sitka)	97	29	25	204	99	41	302	598
Lone Tree Creek	66	18	39	72	124	64	107	461
Bear Creek (Ashland)[a]	75	33	18	196	75	30	290	594
Snake Creek (South of Sitka)	131	51	25	329	91	41	487	822
Big Sand Creek (North of Englewood)	59	23	277	85	99	457	126	822
John's Creek	67	19	203	54	79	334	80	649

[a] Sampled December, 1925.

The field specimens were 10 by 10 by 30-in. blocks reinforced with four light rods in each corner, and an eyebar in the center. Some of the specimens were cast in the laboratory and the balance in the field, using the local sand and gravel aggregate which occurs at the site. The field specimens ranged from 1:3 to very lean mixes, such as were used in the construction of some of the badly disintegrated structures. Table II gives the data on specimens planted at Lakin, and is typical of the specimens placed at each station.

The portland cement used in making the laboratory specimens was a standard brand purchased from the local dealer. It satisfactorily passed the A.S.T.M. standard tests for portland cement. The lumnite cement was furnished by the Atlas Lumnite Cement Co. The cement used in the field specimens was a standard brand taken from the respective county warehouses. The Blue River sand used in the laboratory specimens was a siliceous sand of excellent quality showing a strength ratio of 148; the stone, a local limestone, having a specific gravity of 2.30, with loss by abrasion of 9.7 per cent. This stone satisfactorily passed the sodium-sulfate soundness test as specified by the American Association of State Highway Officials. It is from the same ledge as the limestone used in Series *B* of the freezing and thawing tests.

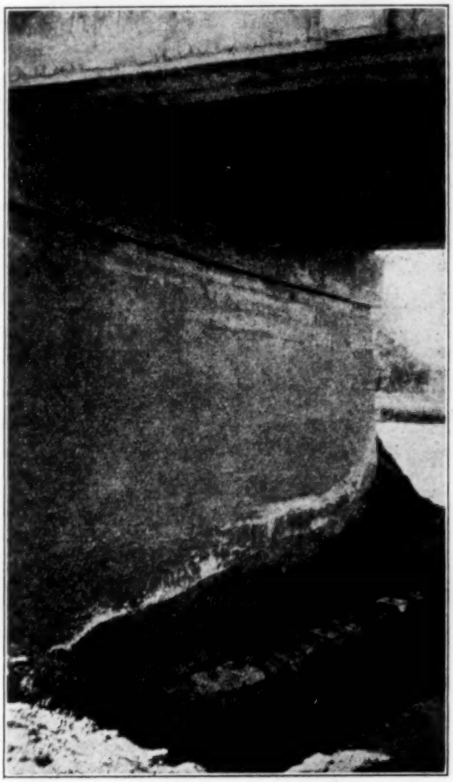

FIG. 3.—Shows Field Specimens **Planted in** Arkansas River at Lakin.

These specimens were planted in September, 1925, and the last notes were taken at an inspection made in October, 1927. To that date all of the laboratory specimens made with portland cement showed no evidence of any action of any kind. Specimens Nos. 20

and 30 of the lumnite specimens show little action other than crazing of the surface; No. 40 was disintegrating rapidly, in places softened to a depth of 2 in.

Of the field specimens, the 1:7 and 1:9 mixes are disintegrating rapidly, the softening being 4 in. deep in some places. The 1:3 and 1:5 specimens show no action. The 1:7 and 1:9 specimens showed evidence of some disintegration at the end of the first winter.

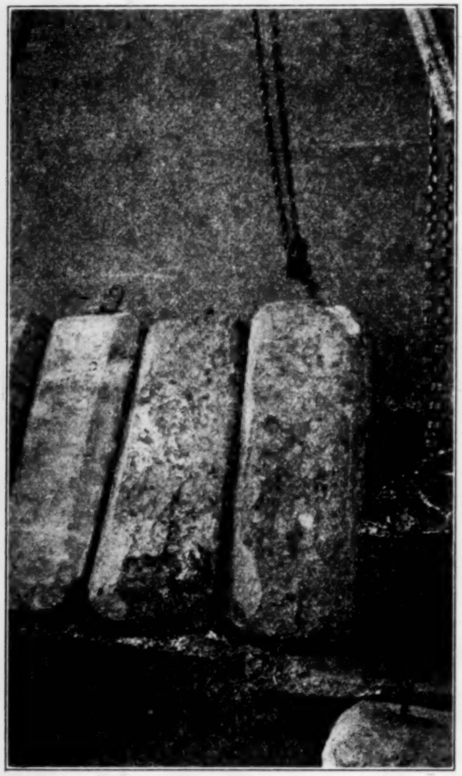

FIG. 4.—Shows the Condition of 1:7 and 1:9 Concrete Two Years After Exposure.

LABORATORY INVESTIGATIONS

Although the failures when called to the attention of the laboratory were considered as due to alkali water, it was early decided to abandon this field of investigation. This particular field is very well covered by able investigators and, further, the preliminary field studies indicated that it was not likely that alkali action had been other than a minor contributing factor.

Two lines of investigation are being carried on with the hope that either or both together may yield some data on the relative durability of concrete:

(*a*) The actual resistance to freezing and thawing action.
(*b*) The relative permeability of the concrete under pressure.

TABLE II.—DATA ON FIELD SPECIMENS AT LAKIN BRIDGE, KEARNY COUNTY.

Specimen	Description	Cement	Mix	Slump, in.	Water-Cement Ratio	Aggregate
No. 5..........	Laboratory	Portland	1:2:4	2	0.70	Blue River sand and limestone
No. 10..........	Laboratory	Portland	1:2:4	3	0.86	Blue River sand and limestone
No. 15..........	Laboratory	Portland	1:3	$2\frac{3}{4}$	0.70	Blue River sand
No. 25..........	Laboratory	Portland	1:4	$2\frac{1}{4}$	0.90	Blue River sand
No. 35..........	Laboratory	Portland	1:6	$1\frac{3}{4}$	1.33	Blue River sand
No. 20..........	Laboratory	Lumnite	1:3	$2\frac{1}{4}$	0.64	Blue River sand
No. 30..........	Laboratory	Lumnite	1:4	2	0.97	Blue River sand
No. 40..........	Laboratory	Lumnite	1:6	$1\frac{3}{4}$	1.25	Blue River sand
No. 3-v..........	Field	Portland	1:3	..	[a]	Arkansas River gravel
No. 5-v..........	Field	Portland	1:5	..	[a]	Arkansas River gravel
No. 7-v..........	Field	Portland	1:7	..	[a]	Arkansas River gravel
No. 9-v..........	Field	Portland	1:9	..	[a]	Arkansas River gravel

[a] Water used in the field mix was the maximum that could be used without having free water on the surface of specimens. The aggregate was secured from under water at the bridge site and measured inundated and the exact water ratio not determined.

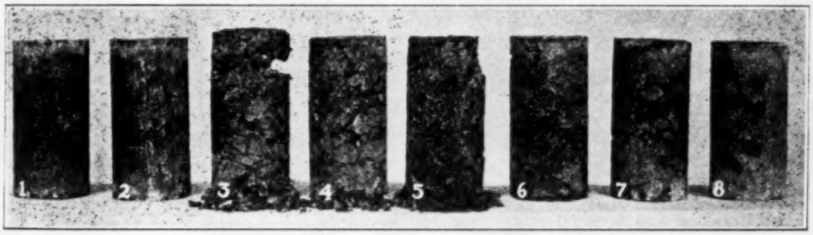

FIG. 5.—Series A of Freezing and Thawing Tests.

Concrete made from 4 different ledges in a large commercial quarry. Proportion of mix, 1:2:4. Pairs of cylinders 1 and 2, 3 and 4, 5 and 6, and 7 and 8 respectively representing the four ledges. The ledge represented by pairs of 1 and 2 showing little unsoundness under the sodium-sulfate test.

Only the results of the freezing-and-thawing tests can be presented at this time, but it is hoped that the results of these two investigations may develop some interesting correlation.

Tests of Concrete Cylinders—Series A:

The first freezing equipment used was a home-made affair consisting of a very well insulated 10-gallon can in which was placed a smaller can around which an ice and salt mixture was packed. With this, a temperature of −3 to −5° F. could be maintained over night. The concrete cylinders shown in Fig. 5 were tested in this box. This series represents a test of stone from four of the principal ledges in a large commercial quarry operating in Kansas. The structure shown in Fig. 1 was built from this stone. Although these ledges proved to be unsound in the sodium-sulfate test and in freezing-and-thawing tests, the producer felt sure that when placed in a good quality of concrete no difficulty would occur. The cylinders shown are 1:2:4 concrete. Four ledges are represented, two cylinders representing each ledge. Cylinders 1 and 2 were from a ledge showing practically no distress in the common sodium-sulfate test. The other ledges showed very marked failure. The condition shown in the illustration is the result of 80 freezing-and-thawing cycles. Cylinders 3, 4, 5 and 6 were completely broken down, with the mortar apparently in good condition around the softened stones. Cylinders 1 and 2 were given about eighty more cycles and the test abandoned, with the concrete of one uninjured and the second destroyed.

Mechanical Refrigerating Apparatus:

This method of testing was so slow, only one reversal a day being practicable, that a special mechanical refrigerating plant was secured. This was purchased from the S. A. Long Co. of Wichita, Kan., the local representative of the Delco Light Co. The equipment is a standard Frigidaire compressor and coil designed for ice cream storage work with a specially built storage box of about 7 cu. ft. capacity. It is divided into two compartments. As the machine now operates, the motor runs until a temperature of about −32° F. is reached. The compressor does not, however, have sufficient capacity to maintain this temperature when the box is being opened and closed and the freezing chambers charged with concrete, so that the ordinary operating temperature is −15 to −20° F. with the compressor operating continuously.

A preliminary series of tests was run to determine the rate of freezing and of thawing. Leeds and Northrup resistance thermometers having a range from −50 to +120° F. were used to determine the temperatures of the ice box and of the cylinders. The 6 by 12-in. concrete cylinders were cast with a hole reaching to the center. The bulb of the resistance thermometer was placed in this hole, tightly packed with asbestos and coated with heavy grease.

Early tests indicated that it was difficult to secure uniform and rapid freezing results when the cylinders were placed in the freezing chamber in the air. Cylinders in the corners froze much faster than those in the center. The larger chamber of the box was then filled half full of alcohol and the concrete cylinders placed in a close fitting corrugated can which was then placed in the alcohol mixture. The space between the can and the cylinder was then filled with water. This gave very rapid results. The alcohol further acted as a sort of storage reservoir, absorbing the heat from the concrete very rapidly and transferring it to the brine. Fig. 6 shows graphically the rate of temperature change for several different conditions of exposure in the freezing chamber. It will be noted that the most rapid results were secured when the specimens were placed in a corrugated can of water and the can immersed in the alcohol. This procedure has been followed with all 6 by 12-in. cylinders. The smaller cylinders have been placed in the can without water and the can placed in the alcohol mixture.

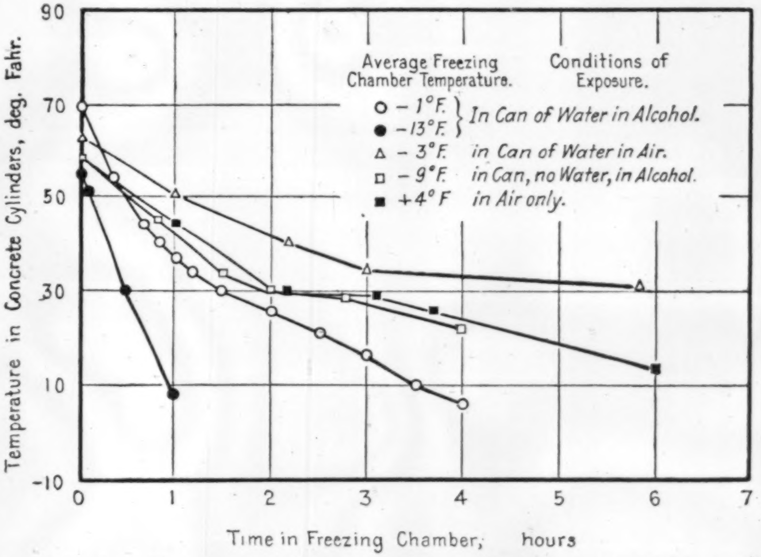

FIG. 6.—Showing Time Required to Freeze 6 by 12-in. Concrete Cylinders Under Various Conditions of Exposure.

Tests as to the thawing time required indicate that 6 by 12-in. cylinders placed in water at 65 to 70° F. will thaw in about one hour.

Our present practice is to make three changes per day, with the freezing temperature at from −15 to −25° F. and thawing for 1½ hours at 65 to 70° F.

The tests to determine the rate of freezing and thawing were made on concrete only 28 days old, and results indicated that it would be better to use concrete that had been allowed to age for a longer period of time. The first series of mortar specimens to be tested for loss in strength were started at 28 days and results were very erratic, cylinders sometimes gaining strength until they fell apart. As a result, the general plan now is to start no tests at less than 90 days.

Tests of Concrete of Varying Water-Cement Ratio—Series B:

The first carefully controlled specimens carried through to destruction in the refrigerating machine were 3 by 8-in. cores cut from concrete about one year old. This concrete had been made up for another series of experiments.[1] The concrete was proportioned 1:2:3½ with

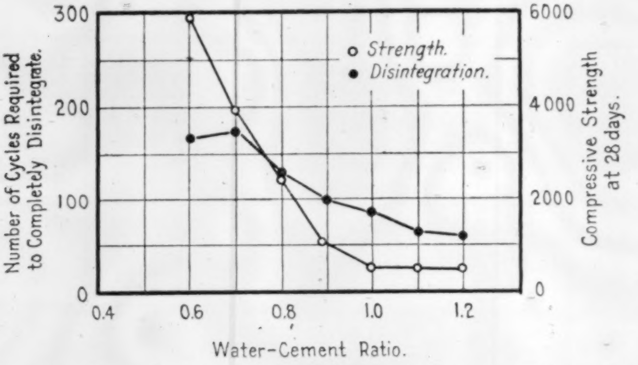

FIG. 7.—Tests on 3 by 8-in. Cores Cut from 1:2:3½ Concrete Having Variable Water-Cement Ratio.

Frozen at −15 to −25° F. Thawed at 70° F.

water-cement ratios varying from 0.6 to 1.2. The sand used was a local sand having a fineness modulus of 3.12. The limestone, a local limestone of fair quality, satisfactorily passed the sodium-sulfate test. The cores were frozen to complete disintegration with the results shown in Fig. 7.

Examination of these specimens revealed several items of interest. Considerable difficulty was encountered in determining just when a specimen had disintegrated or when two different specimens had reached the same stage of disintegration. The values reached and used in plotting Fig. 7 were determined largely by trial and when a specimen had reached a point where it crumbled in the hand to a third or less of its original size, it was considered as having completely

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[1] "Wear Test of Concrete," *Bulletin No. 20,* Engineering Experiment Station, Kansas State Agricultural College.

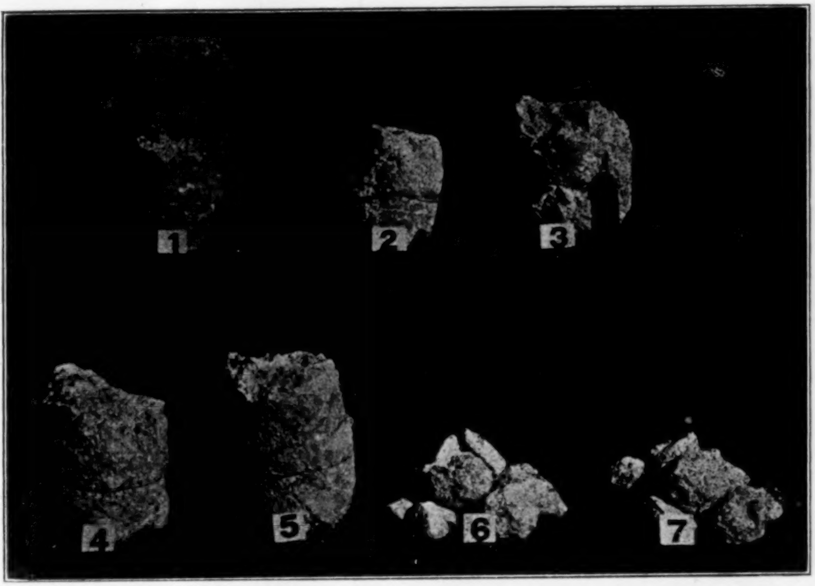

FIG. 8.—Series B, Tests of 1:2:3½ Concrete with Varying Water-Cement Ratios.

Numbers 1, 2, 3, 4, 5, 6 and 7 representing water-cement ratio of 0.6, 0.7, 0.8, 0.9, 1.0, 1.1, and 1.2 respectively.

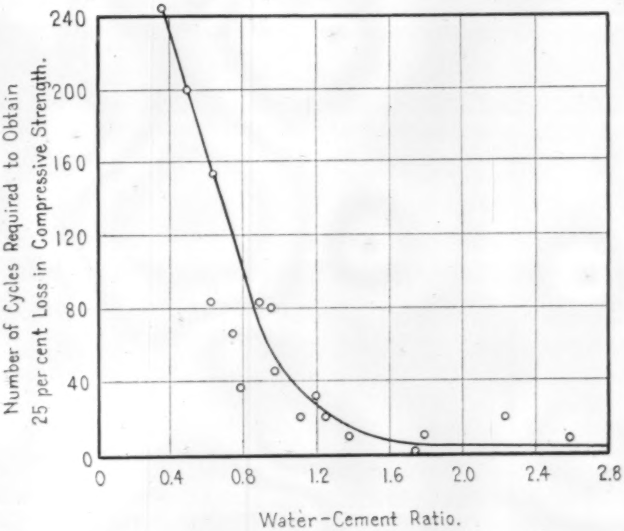

FIG. 9.—Curve Showing Number of Cycles Required to Give 25 Per Cent Loss in Strength for a Given Water-Cement Ratio.

Mortar specimens of normal consistency, 2 by 4 in. Frozen at −15 to −20° F. Thawed at 70° F.

disintegrated. The core representing the water-cement ratio of 0.6 has not yet quite reached this stage. (See Fig. 8.)

In this series, concrete having a water-cement ratio of 0.9 or more failed distinctly by a mortar disintegration. At a water-cement ratio of 0.9 some of the stone began to show signs of failure, and for all of the lower ratios the disintegration of the stone is playing a very important part; in fact, most if not all, of the failure in the specimen of 0.6 water-cement ratio appears to have been due to failure of the coarse aggregate.

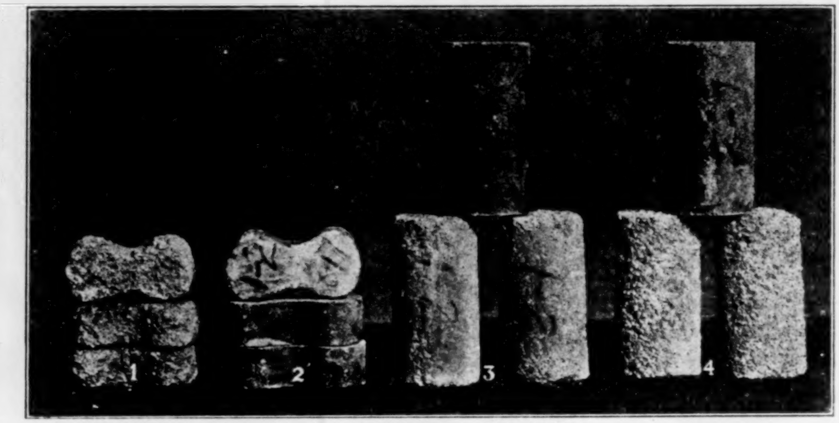

FIG. 10.—Series C of Freezing and Thawing Tests.

River-sand and flint-sand mortars. Nos. 1, 2 and 4 represent flint sand, and No. 3, river sand.

Tests of Mortars of Several Proportions—Series C:

This series consisted of 2 by 4-in. cylinders of mortar of constant consistency or workability as determined by the judgment of a skilled operator. Mixes were arbitrarily chosen of 1:2, 1:3, 1:4, 1:5, 1:7, and 1:9. Several series like this have been started using river sand, bank sand, and crushed flint sand. In view of the difficulty in interpreting values for total disintegration, an effort is being made to determine the point where the compressive strength has been reduced an arbitrary percentage. We are at present using a loss of 25 per cent as a basis. The values as given by results to date are shown in Fig. 9.

This method seems to hold considerable promise, but certain very erratic results are occurring with the rich mortars, especially with the harsh working sands like crushed flint. Here we have experienced an apparent growth in strength after sufficient freezing-and-thawing cycles have been completed to very markedly affect the surface.

The relative appearance of river sand and flint sand mortars are shown in Fig. 10. All of these specimens are 1:2 mortar. Specimens Nos. 1 and 2 show tension briquets of flint sand before and after freezing 181 times. Specimen No. 3 shows river sand cylinders before and after freezing 284 times and No. 4, the crushed flint sand, after 181 cycles.

Figure 11 shows the comparative condition of a mortar series when all have been given 25 cycles. This is one of the groups in Series C.

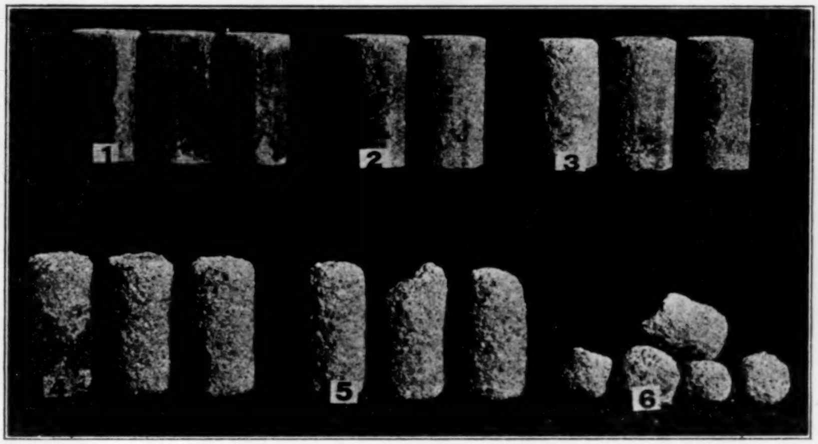

FIG. 11.—Showing Effect of 25 Cycles of Freezing and Thawing on Mortar of Varying Proportion.

See text for proportions and water-cement ratios.

The proportions and water-cement ratios are shown below:

CYLINDERS	MIX	WATER-CEMENT RATIO
No. 1	1:2	0.58
No. 2	1:3	0.8
No. 3	1:4	1.15
No. 4	1:5	1.39
No. 5	1:7	1.8
No. 6	1:9	2.64

In general the results of such an investigation as this are very difficult to present except in a graphic way. Photographs have been very largely used in recording the results, but only a few of these can be included in this paper.

CONCLUSIONS

Insufficient data have been secured to date from which to draw definite conclusions, but the data thus far secured leads the author to believe that the following conclusions are indicated:

1. That the alternate freezing of saturated concrete at low temperatures and thawing at room temperatures is a valuable means of studying the durability of concrete and concrete aggregates and is worthy of further study.
2. That variations in the water-cement ratio of plastic workable concrete has a very marked effect upon its durability.
3. That a water-cement ratio of 1.0 is near the maximum permissible for a desirable durability of concrete in exposed situations.
4. That the use of unsound aggregate produces unsound concrete, the resistance of the mortar to disintegration being only slightly effective in protecting the aggregate.
5. That the disintegration of concrete through the failure of the mortar is a surface action and the outside is very badly disintegrated before the interior is affected.
6. That the physical characteristics of the mortar after failure are such as to give the impression of chemical or solution action.
7. That the results secured are comparative only, and that it is not feasible to translate the results into actual length of life under natural conditions.

Acknowledgments:

The author wishes to acknowledge the assistance of the staff of the Kansas Highway Commission, and his associates in the Engineering Experiment Station at the Kansas State Agricultural College; also the advice and assistance of Mr. A. A. Anderson of the Portland Cement Association.

DISCUSSION

Mr. Freeman.

MR. P. J. FREEMAN[1] (*presented in written form*).—It was only five years ago that the Society held a Symposium on "What Properties of and Methods of Making Concrete Require Further Investigation?"[2] Since that time the subject of durability of concrete has been gradually occupying larger space in our technical journals and in the discussions of engineering societies. Several of the papers presented at this annual meeting emphasize the fact that strength is only one of the things to be considered in concrete construction. A few years ago strength was over-emphasized and very little attention was paid to a study of the durability of concrete.

It is necessary to study causes affecting the durability of concrete from all angles and to-day this is being done. Excellent work is going on at the U. S. Bureau of Standards on portland cement and the papers presented at this meeting show that other materials also are being given serious study.

It is very necessary that serious consideration be given this subject of durability of concrete. The annual market value of concrete products made from portland cement is at least twice the market value of all the pig iron produced in the United States during the same period, or about one-and-one-half billion dollars. If only 10 per cent of this concrete is defective from any cause the country suffers a huge economic loss and therefore methods used improving concrete merit unlimited study.

Mr. Scholer has pointed out a way for making one particular phase of investigation and the methods discussed will help other investigators to eliminate much preliminary work. The effect of rate of freezing is one of the outstanding features in his paper, and undoubtedly this explains some of the erratic results secured by other laboratories.

The apparatus used is simple and apparently the equipment is much better than experimenters have used before and it is to be hoped that Mr. Scholer will continue to study this important phase of a problem which is common to all parts of this country where freezing weather occurs.

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[1] Chief Engineer, Bureau of Tests and Specifications, Department of Public Works, Allegheny County, Pittsburgh, Pa.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 156 (1923).

Mr. Goldbeck.

MR. A. T. GOLDBECK[1] (*presented in written form*).—Mr. Scholer has begun tests in a field which has long been neglected by concrete investigators. Intensive work on durability and the factors which affect it cannot but result in the improvement of concrete as a structural material. We have been too persistent in focusing our attention on the compressive strength of concrete and have failed to realize the importance of other properties which concrete should possess, and among them is that of durability under long-time exposure.

There are many factors which control the durability of concrete and Mr. Scholer, admittedly, has merely made a beginning in the study of these variables. Ultimately, we must devise a test for durability of concrete and of the materials composing it which will be reasonably indicative of service results. Such a test probably will be one of varying intensity, depending upon the general climatic conditions which it is intended to simulate. This is most important, for undoubtedly it is possible to break down almost any concrete by subjecting it to a large number of severe freezing cycles.

It is interesting to note that the stone used by Mr. Scholer in his specimens shown in Fig. 8 of his paper withstood the sodium sulfate test and yet showed disintegration in the freezing and thawing test of concrete. Ordinarily, the sodium sulfate test is considered as being more severe than service action, and consequently the question may well be raised as to whether the freezing test method used by Mr. Scholer is not more severe than encountered in service. Possibly the speed at which the specimens are reduced to freezing temperatures has an important effect. The limestone used in Mr. Scholer's tests was below the average of limestones generally regarded as acceptable aggregates, and it is important that freezing tests be made on concrete using limestone and other aggregates of better quality in order that the border line of acceptability may be discovered.

Mr. Scholer is to be commended for his initiation of a program of work on the durability of concrete and the various factors affecting durability, and it is hoped that his work will serve as a stimulation to others to engage in work of a like nature.

Mr. Miner.

MR. J. L. MINER.[2]—I want to take this opportunity to compliment Mr. Scholer on the very interesting work he is doing and the valuable results he has obtained. Last fall I spent several days with him, both in the field and in the laboratory, examining specimens and reviewing his results. One feature of his work seems to be especially significant and I wish to refer to it.

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[1] Director, Bureau of Engineering, National Crushed Stone Association, Washington, D. C.

[2] Technical Manager, The Atlas Lumnite Cement Co., New York City.

Mr. Miner. Examination of the specimens showed that the mortar fails first on repeated freezing and thawing in those specimens made with the higher water-cement ratios. On the other hand, the aggregate fails first on repeated freezing and thawing in those specimens made with the lower water-cement ratios.

This is very interesting since it indicates that the amount of mixing water has a decided effect on the character of the products of hydration, and I believe that an excess of mixing water not only lowers the strength of the cementing media but also reduces its resistance to the chemical action of moisture.

There is one point that was not brought out in Mr. Scholer's paper which has a bearing on the results he has reported. Specimens Nos. 30 and 40 were cured during warm weather under wet burlap. Mr. Mattimore, in his paper before this Society three years ago,[1] showed that the curing of lumnite-cement concrete under wet burlap affected the rate of hydration and appreciably lowered the strength of the concrete. Observation of results, both in the field and in the laboratory, leads me to the conclusion that the curing of lumnite mortar or concrete under wet coverings of any kind not only lowers the strength but also very materially lowers the resistance of the concrete to the subsequent action of moisture and frost. It is my belief that the present condition of these particular specimens is the result of the effect of curing conditions on the nature of the cementing media.

Mr. Scholer. MR. C. H. SCHOLER.[2]—I wish to assure Mr. Goldbeck that I consider our freezing-and-thawing tests at this rapid rate to be very severe, possibly as severe as the sodium sulfate test. We have a number of stones which have stood up under the sodium sulfate test that break down under the freezing-and-thawing test at comparatively early ages.

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[1] H. S. Mattimore, "A Study of Temperatures in High-Alumina Cement and Methods of Curing," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 192 (1925).

[2] Professor of Applied Mechanics and Engineer of Tests, Kansas State Agricultural College, Road Materials Laboratory, Manhattan, Kans.

PERMEABILITY OF CONCRETE

By Ira L. Collier[1]

Synopsis

The paper describes a method of determining the flow of water through concrete, the water being under a pressure of 145 lb. per sq. in. Three series of experiments are reported. In the first series, all factors were constant except the water-cement ratio of the concrete, which varied from 0.6 to 1.2. In the second series, all factors were constant, including the water-cement ratio, except the slump which varied from a dry mix to a wet mix. In the third series, water-cement ratio and slump were constant, but the percentage of sand and gravel was varied, giving variable fineness moduli.

These tests have shown: (1) a general straight-line relationship between flow of water through concrete and water-cement ratio, when the data are plotted to logarithmic scale; (2) that the flow decreased as the cement content was increased, although no definite relationship has been established; and (3) that the flow decreased with decrease in fineness modulus.

The suggestion for making this apparatus for determining the permeability of concrete described in this paper was obtained from the University of Wisconsin. The apparatus was made from steel pipe and fittings, and Fig. 1 shows the detail of a single holder. This figure will be referred to later when describing the method of operation.

Materials and Specimens

Cement.—Ten sacks of "Superior" brand cement were spread out on a clean concrete floor and thoroughly mixed by hand, then stored in metal containers. Tests made by students indicated that the cement was normal in all respects.

Sand.—All sand was dried and separated on Tyler screens. The different sized particles were then re-combined in the following proportions: Material retained on No. 4 sieve, 0 per cent; on No. 14 sieve, 11 per cent; on No. 28 sieve, 20 per cent; on No. 48 sieve, 66 per cent; and on No. 100 sieve, 3 per cent. The fineness modulus was 2.39.

Gravel.—All gravel was dried and separated on screens having circular openings. The different sized particles were then re-combined in the following proportions: Material retained on 1½-in. screen, 0 per cent; on 1-in. screen, 11 per cent; on ¾-in. screen, 19 per cent;

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[1] Assistant Professor of Civil Engineering, University of Washington, Seattle, Wash.

on ½-in. screen, 40 per cent; on ¼-in. screen, 25 per cent; passing ¼-in. screen, 5 per cent. The fineness modulus was 6.94

Proportions, Mixing, and Curing.—Thirteen pounds of sand and 26 lb. of gravel, graded as stated above, were used in the first two series. Specimens were 9 in. in diameter and 8 in. high. In the first series, the variable studied was the water-cement ratio. The water and cement were mixed in the desired ratio and enough of this slurry was added to the sand and gravel, which had been first mixed thoroughly, to produce a concrete having a consistency measured by a 4-in. slump. The water-cement ratios studied in this series varied from 0.6 to 1.2, increments being even tenths.

TABLE I.—TEST DATA ON PERMEABILITY OF CONCRETE.

All values of flow are average of 5 tests, unless otherwise indicated.
Specimens 1 to 9 were made by L. A. Carter, a research student; the others were made by the writer.

Series	Set. No.	Fineness Modulus	Proportions by Weight			Water-Cement Ratio	Slump, in.	Density	Weight, lb. per cu. ft.	Quantity of Materials per cu. yd. of Concrete			Flow, gal. per hr. at end of 100 hr.	Mean Error, per cent	Condition of Surface at end of 100 hr.
			Cement	Sand	Gravel					Cement, bbl.	Sand, lb.	Gravel, lb.			
I	1	5.41	1	1.14	2.28	0.6	4	0.853	145	2.39	1020	2040	0.000063	31	Dry
	2	5.41	1	1.66	3.32	0.7	4	0.861	...	1.77	1100	2200	0.000071	17	Dry
	3	5.41	1	2.22	4.44	0.8	4	0.856	...	1.37	1140	2280	0.0134[c]	35	Wet
	4	5.41	1	2.82	5.64	0.9	4	0.850	...	1.12	1160	2320	0.00064	59	Dry
	5	5.41	1	3.02	6.04	1.0	4	0.842	...	1.02	1160	2320	0.0091	47	Wet
	6	5.41	1	3.52	7.04	1.1	4	0.830	...	0.87	1150	2300	0.0379[b]	74	Wet
	7	5.41	1	3.76	7.52	1.2	4	0.816	...	0.80	1140	2280	0.049[b]	69	Wet
II	8	5.41	1	3.26	6.52	0.9	0	0.790	140	0.89	1090	2180	0.406[a]	65	Wet
	10	5.41	1	2.81	5.62	0.9	½	0.793	140	1.04	1070	2140	0.0659[b]	57	Wet
	11	5.41	1	2.48	4.96	0.9	2	0.813	149	1.18	1080	2160	0.00138	52	Wet
	12	5.41	1	2.21	4.42	0.9	6	0.819	148	1.30	1080	2160	0.00026	27	Dry
	9	5.41	1	2.00	4.00	0.9	8	0.811	147	1.41	1070	2140	0.00095	32	Wet
III	13	6.03	1	1.50	6.00	0.9	4	0.840	150	1.20	680	2710	0.00082	27	Wet
	14	5.56	1	1.93	4.51	0.9	4	0.801	147	1.31	950	2140	0.00063	54	Wet
	15	5.11	1	2.27	3.40	0.9	4	0.782	145	1.44	1220	1840	0.00030	26	Dry
	16	4.66	1	2.46	2.46	0.9	4	0.754	141	1.56	1440	1440	0.00077[d]	36	Wet

[a] Average of four tests.
[b] Average of three tests.
[c] Specimens were three days late in being placed in curing water.
[d] Specimens were one day late in being placed in curing water.

In the second series the consistency of the concrete was varied while the water-cement ratio was held constant at 0.9. Enough slurry was added to the sand and gravel first to give a zero, then ½-in., 2-in., and 8-in. slump.

In the third series the water-cement ratio was held at 0.9, the slump 4 in., and the relationship between the sand and gravel was varied as follows: 20:80, 30:70, 40:60, and 50:50.

The proportions water-cement ratio, slump, density, and other properties of the concrete in these three series are given in Table I.

The time of mixing for all specimens was 1½ minutes. All concrete was placed in the molds in three layers, each layer being tamped twenty-five times with a standard 24-in. bullet-pointed ⅝-in. round bar. Five specimens of like conditions were made on the same day. The next day the specimens were taken from the molds and stored in water the temperature of which varied between 65 and 75° F.

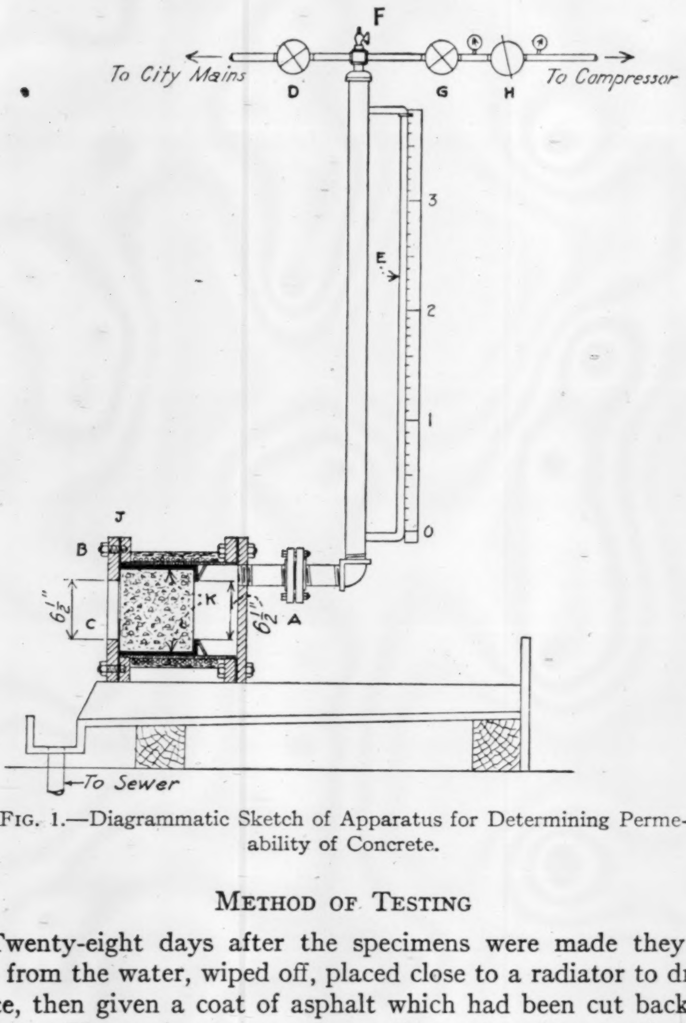

FIG. 1.—Diagrammatic Sketch of Apparatus for Determining Permeability of Concrete.

METHOD OF TESTING

Twenty-eight days after the specimens were made they were taken from the water, wiped off, placed close to a radiator to dry the surface, then given a coat of asphalt which had been cut back with kerosine. The testing apparatus was then set up in a vertical position, the companion flanges *A*, Fig. 1, having been disconnected. The specimens were then centered in the 10-in. opening. Asphalt just hot enough to pour easily was then poured in the ½-in. opening around the specimen, at *K*. In order that the specimen would have a good bearing against the outer flange *B*, it was necessary to cap that portion of the specimen with some quick setting material. A mixture of 2 parts lumnite cement and 3 parts portland cement was found to set hard in 15 minutes. A ring of this material, *J*, mixed with water to a stiff paste, was placed on top of the asphalt. The outer flange, well oiled, was then set in place. The cement that squeezed out into the opening *C* was cut away after hardening. The holder was then lowered to a horizontal position and the companion flanges *A* connected. Valve *D* was then opened and the apparatus filled with water to the top of the gage *E*. Air was allowed to escape from petcock *F*. The next morning all the nuts to the outer flange were tightened (this was found necessary, as the slightest movement of the specimen would break the asphalt seal) and the air pressure turned on, valve *G*. The air pressure was held between 143 and 148 lb. per sq. in. by means of the reducing valve *H*. The compressor held the pressure between 200 and 240 lb. per sq. in.

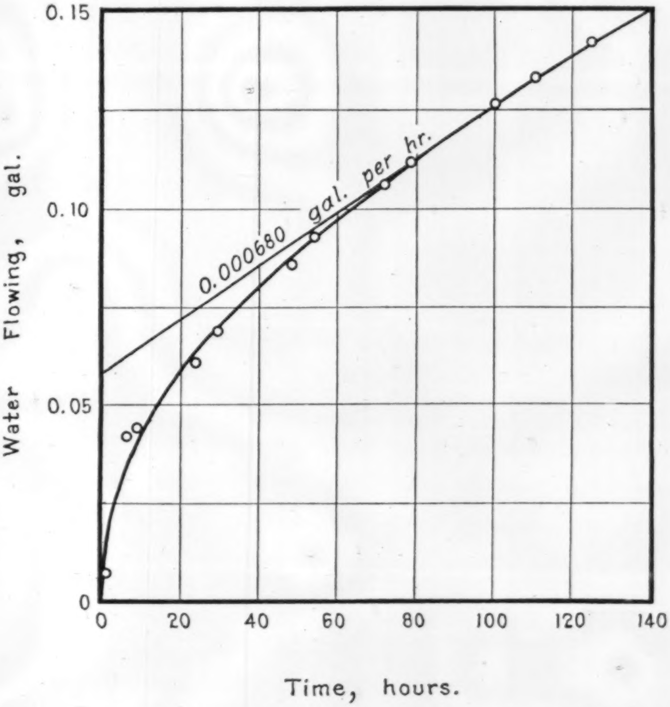

FIG. 2.—Typical Flow Curve (Specimen 1, Set 9, Second Series).

Readings of the gage were then taken for six days, at which time a new set of specimens was ready. The frequency at which the readings were taken depended on the rate of flow, never less than two readings a day. In some cases where the rate of flow was high, it was necessary to refill the columns every few hours. In these cases the pressure was cut off during the night.

To remove the specimens from the holder, electric current was applied to the coil of resistance wire wrapped around the pipe. The wire was insulated from the pipe by means of a sheet of mica and in order that the heat would not be dissipated, the outside of the pipe and coil were given a thick coat of insulating material. A sheet of galvanized iron then protected this material. The outer flange was then taken off and in a few minutes the weight of the column of water would force the specimen out. The apparatus was then ready for a new set of specimens.

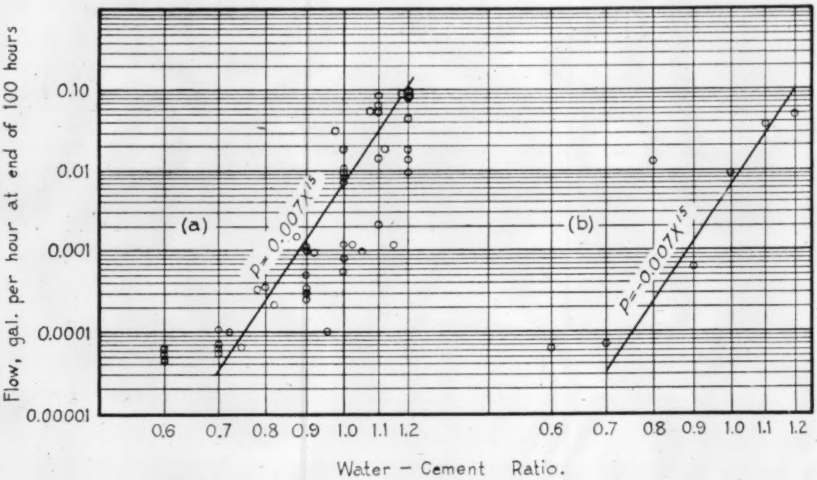

FIG. 3.—Illustrating the Relation Between Permeability and Water-Cement Ratio (First Series).

TEST DATA

The apparatus having been calibrated, the gage readings were reduced to gallons and plotted against the hours of elapsed time. A smooth curve was then drawn through the points and a tangent to the curve drawn at the 100-hour ordinate. The slope of the tangent gave the rate of flow in gallons per hour at the end of 100 hours. This value for all specimens was then taken as the unit for the study of the permeability of concrete and will hereafter be spoken of as "the flow." A typical plot is shown in Fig. 2.

The average values of flow, as thus determined, for the specimens of the three series are given in Table I. Except where noted, each value of flow reported is the average of five tests.

Values of flow for individual specimens have been plotted against water-cement ratio to logarithmic scale in Fig. 3 (*a*), the data being taken from the specimens of the first series as well as from specimens in a preliminary series. A straight line drawn to average the points thus plotted has the equation $P = 0.007\ x^{15}$, in which P is the flow and x is the water-cement ratio. In Fig. 3 (*b*) are plotted the average flows as determined for the specimens of series 1 and reported in Table I. With the exception of the value of flow of 0.0134 gallon per hour for tests No. 3, water-cement ratio 0.8, the straight-line rela-

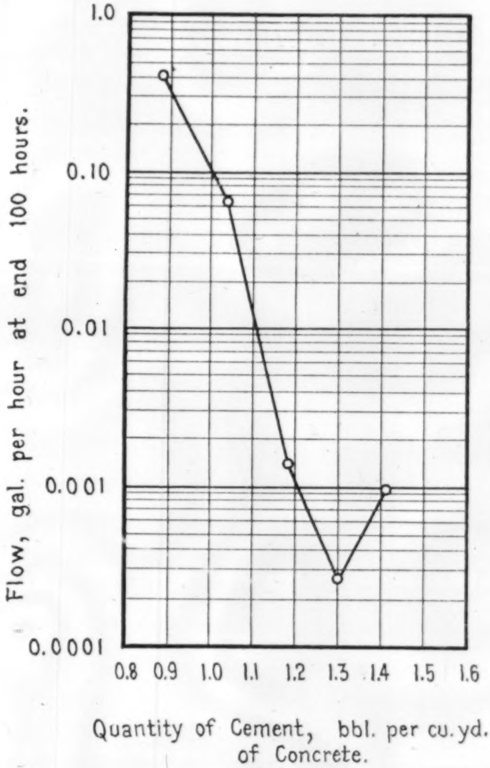

FIG. 4.—Illustrating the Relation Between Permeability and Quantity of Cement (Second Series).

tion as expressed by the above equation is fairly representative of these data.

In this series specimens with the water-cement ratio of 1.0, 1.1 and 1.2 leaked water at all times; specimens with a water-cement ratio of 0.9 showed water on the surface for the first two days only, after which time the water evaporated as fast as it came through. The richer mixes showed no sign of water on the surface at any time. It is probable that the high value of flow for the specimen of 0.8 water-cement ratio has been caused by improper curing of the specimens of that set since they were three days late in being placed in curing water.

The test data from the second series, in which the consistencies of the concrete were varied, have been plotted in Fig. 4. The specimens containing 0.89 and 1.04 bbl. of cement per cubic yard of concrete would be classed as unworkable. More data taken from specimens in the plastic or workable zone are needed before definite conclusions can be drawn. However, it appears that in general flow is decreased as the cement content has been increased, although no definite relationship has been established.

The test data from the third series have not been plotted; but it may readily be seen from Table I that, except for the last set of tests, No. 16, fineness modulus 4.66, the flow through the concrete is decreased almost directly in proportion to increase in percentage of sand. It was found that the specimens of set No. 16 were one day late in being placed in curing water and this fact, in conjunction with the high flow reported for set No. 3, which had also been improperly cured, emphasizes the necessity for the complete curing of concrete which is to be subjected to high water pressures.

DISCUSSION

Mr. Johnson.

Mr. N. C. Johnson.[1]—The question of impermeability is one of paramount importance to a large part of the concrete industry. Impermeability is of even greater importance than is strength. If we have impermeability we may expect, under average conditions, a high degree of permanence. The same does not hold for strength and particularly there seems to be no relation between high early strength and impermeability and indeed between high early strength and a variety of other qualities which are essential and necessary to concrete.

It should be noted that there is a popular misconception as regards the nature of the cement content of concrete and this popular misconception also bears upon the question of impermeability. The products of the hydration of cement are those which we utilize in binding sand and stone together to make concrete. In nature, these hydrated products are largely colloidal.

A colloid is a jelly and the characteristics of every jelly is that it will indefinitely absorb water and distend with water and that it will transfer and transfuse water throughout its mass.

Any detailed examination of concrete in water exposure will show that there is a constant absorption of water by the concrete, which means, of course, an absorption and transfusion of water through the colloid that surrounds the sand and stone particles. This colloid is the derived product, due to hydration of cement with water. This absorption and transfusion of water is therefore a natural quality of concrete; and in the average concrete, the process of absorption and transfusion will continue until eventually the colloid is so distended and so saturated that it will have little or no bonding power *so long as it remains in this saturated condition.*

It is very well worth while to recognize these things, because many laboratory and other investigations with their derived data on concrete are quite misleading as regards the value or lack of value of any concrete, inasmuch as they do not provide and cannot provide for a duplication of those situations and conditions which exist in actual field construction.

We must reconcile ourselves to these things, inasmuch as they are a natural property of concrete. We cannot, no matter how much we talk about it, alter the ultimate nature or constitution of portland cement or its derivatives with water, unless there is a more radical

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[1] Hool and Johnson, Engineers, New York City.

Mr. Johnson. departure from the present composition of cement than is indicated by any work undertaken to date. So long as the products of hydration of cement are colloidal, we will have absorption. And so long as commercial procedures remain as they are, we shall also have other faults entering in, such as the actual passage of the water in porous or badly made concrete, or through those parts of the concrete which are properly termed laitance or muck. The laboratory does not reproduce these things, so that laboratory data alone, not correlated with the field, is very misleading as regards the nature of concrete as we find it and as we must depend upon it in field construction.

There is at present too much concentration of thought on strength and strength alone, and particularly on high early strength, to promise any warrant of an improvement in the art as a whole, or any progress in commercial work towards a better impermeability.

Mr. Bates. MR. P. H. BATES.[1]—It happens that I am unfortunately one of the joint authors of the oldest papers on the subject of the permeability of concrete, and at the time we carried on these experiments, which are now about eighteen years old, one of the biggest difficulties we had was obtaining in the laboratory a concrete that would leak. From time to time since that, we have had numerous inquiries in regard to the waterproofing of concrete and other masonry construction. In more recent times, we have been severely criticized because we have not been able to supply anyone with any more recent publications than this eighteen-year-old one. As a consequence of that we have felt, contrary to our own convictions, that it would be desirable to start some new investigations on the variety, character, usability, etc., of the various and innumerable and not new, as compared with those eighteen-years-old, waterproofing compounds now on the market. Again we have now been spending three months and have not yet been able to produce in the laboratory a concrete that deserves the name of concrete, regardless of the water-cement ratio, that will be permeable.

Mr. Collier. MR. IRA L. COLLIER.[2]—I might say that in all of those specimens with a low water-cement ratio, no water showed on the surface. The first of these specimens with a water-cement ratio of 0.7 showed no water. I measured the water going into the specimen and of course there was water going in at all times, but it did not come to the surface, the amount was so small, as Mr. Johnson says, the colloids probably absorbed it. I do not know how long that would continue probably indefinitely.

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[1] Chief, Clay and Silicate Products Division, U. S. Bureau of Standards, Washington, D. C.

[2] Assistant Professor of Civil Engineering, University of Washington, Seattle, Wash.

THE DETERMINATION OF THE WORKABILITY OF CONCRETE

By Wallace F. Purrington[1] and Harold C. Loring[2]

Synopsis

The paper describes a method of determining the workability of a concrete mixture by measuring the power consumed in mixing the concrete. A concrete mixer is driven by a suitable motor, and the load applied to the mixer measured by a polyphase wattmeter supplied with proper transformers. The method of operation is described and the results of a number of tests are given, which indicate that the method has possibilities for practical applications in the study of this important property of a concrete mixture.

Concrete engineers use the expression "workability" to an ever increasing extent. Its importance in the production of good concrete is being recognized and is the subject of study by many investigators. Nevertheless, it is an elusive term. Its value has been measured by the use of the slump cone, but the authors are not satisfied that this is a measure of the quality of concrete in which we are interested. Gravity enters into the results to too great an extent. The flow table is not wholly satisfactory. The so-called "squeezometer," penetration determinations and the forcing of neat cement pastes through orifices have been studied, but the authors are not satisfied with the results.

Let us strip the term "workability" of all its technical definitions and bring it down to the language of the laborer on the job. He has a mass of concrete before him and a shovel in his hand. His problem is to move the material from one point to another. If the material is very stiff a considerably greater outlay of energy is required than if it is more plastic. He pushes his shovel into the mass and pulls the concrete ahead, involving the principle of shear. Suppose, now, that for the shovel is substituted a piece of iron which is pushed through the mass of concrete, and that we can measure the work done; we should then obtain information of some value.

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[1] Materials Engineer, New Hampshire Highway Department, Concord, N. H.

[2] Assistant Materials Engineer, New Hampshire Highway Department, Concord, N. H.

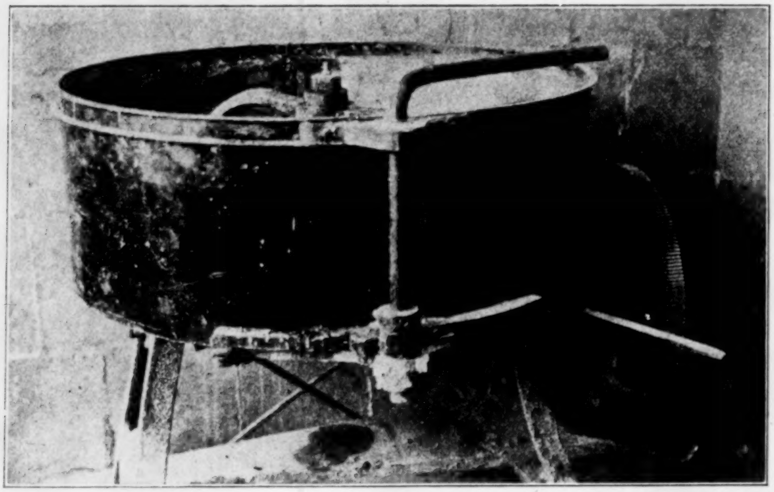
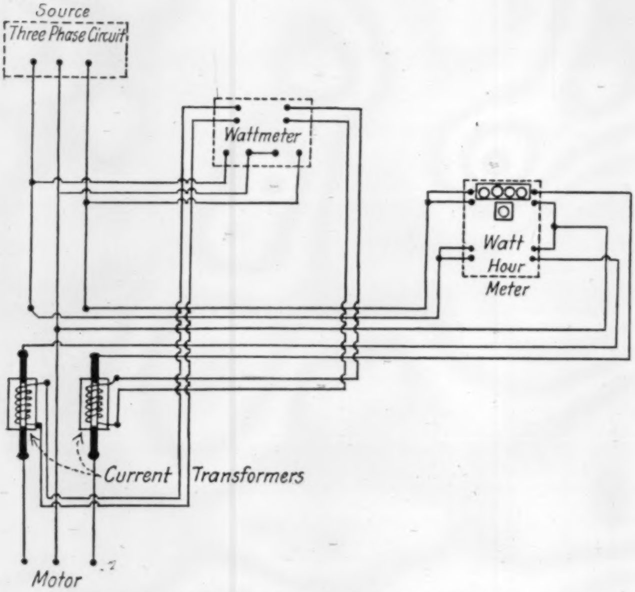
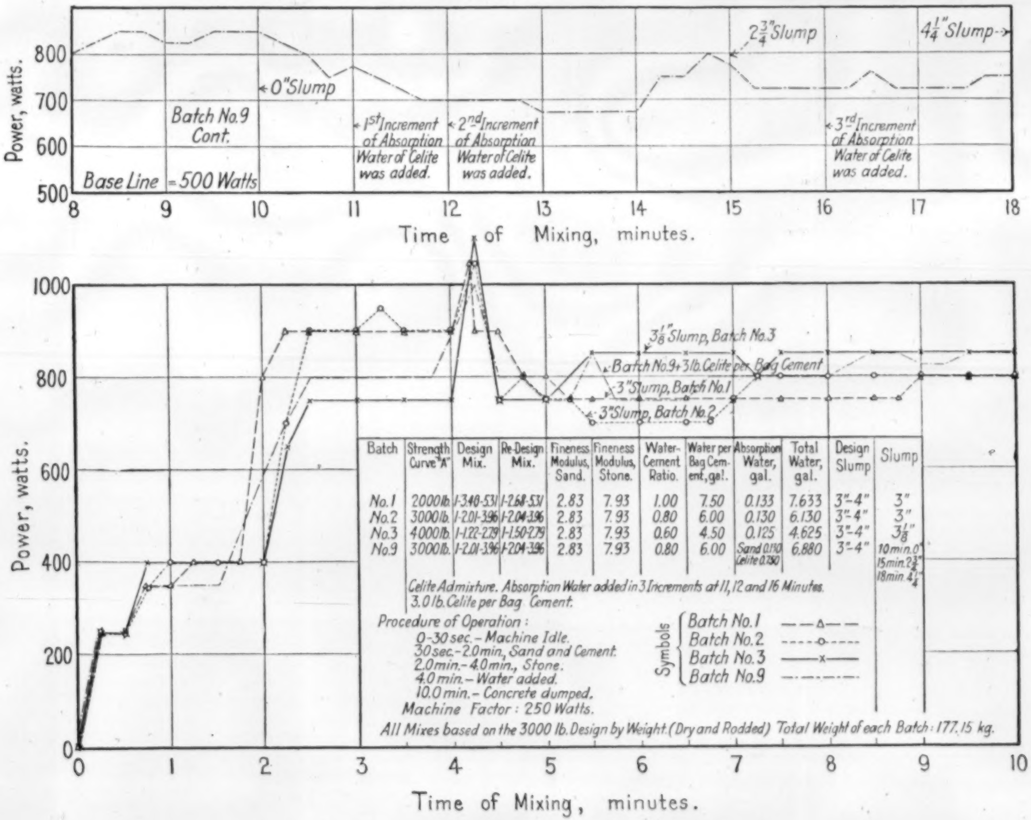


FIG. 1.—Apparatus for Determining Workability by Measurement of Energy Required in Mixing.



Source
Three Phase Circuit
Wattmeter
Watt
Hour
Meter
Current
Transformers
Motor

FIG. 2.—Wiring Diagram for Reading and Recording Devices Used on Testing Apparatus.



Batch	Strength Curve "A"	Design Mix.	Re-Design Mix.	Fineness Modulus, Sand.	Fineness Modulus, Stone.	Water-Cement Ratio.	Water per Bag Cement, gal.	Absorption Water, gal.	Total Water, gal.	Design Slump	Slump
No. 1	2000 lb.	1-3.40-5.31	1-2.68-5.31	2.83	7.93	1.00	7.50	0.133	7.633	3"-4"	3"
No. 2	3000 lb.	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.00	0.130	6.130	3"-4"	3"
No. 3	4000 lb.	1-1.22-2.79	1-1.50-2.79	2.83	7.93	0.60	4.50	0.125	4.625	3"-4"	3⅛"
No. 9	3000 lb.	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.00	Sand 0.130 Celite 0.750	6.880	3"-4"	10 min. 0" 15 min. 2¾" 18 min. 4¼"

Celite Admixture. Absorption Water added in 3 Increments at 11, 12 and 16 Minutes.
3.0 lb. Celite per Bag Cement.

Procedure of Operation:
0-30 sec. – Machine Idle.
30 sec.-2.0 min., Sand and Cement.
2.0 min.-4.0 min., Stone.
4.0 min. – Water added.
10.0 min. – Concrete dumped.
Machine Factor: 250 Watts.

Symbols	
Batch No. 1	— —△— —
Batch No. 2	- - - -○- - - -
Batch No. 3	———×———
Batch No. 9	—·—·—·—

All Mixes based on the 3000 lb. Design by Weight. (Dry and Rodded) Total Weight of each Batch: 177.15 kg.

FIG. 3.—Results of Tests on Four Batches of Concrete.

TESTING APPARATUS

With this thought in mind the New Hampshire State Highway Department recently purchased and installed a half-bag tub-type concrete mixer equipped with paddles which revolve on arms fastened to a movable center post. The paddles just clear the bottom and sides of the tub. They are driven by a $2\frac{1}{2}$-hp. motor running at 1700 r. p. m., so geared that the paddles revolve at the rate of 23 r. p. m. Connected to the motor and attached to the electric line is a polyphase hectowattmeter supplied with the proper transformers which give readings dependent upon the load applied to the mixer. The apparatus is illustrated in Fig. 1 and the wiring diagram is shown in Fig. 2.

The mixer is run without load for a half minute and the wattage is noted. This determines the "machine factor" and under our conditions is 250 watts. Weighed quantities of cement and sand are introduced and the machine runs with this load for $1\frac{1}{2}$ minutes. The coarse aggregate is added and the wattage is noted for 2 minutes. Readings are taken at 15-second intervals. Referring to Fig. 3, it will be noted that after each increment of load the curve becomes a horizontal line as soon as the material is thoroughly mixed. The water is then added and the curve immediately rises to a maximum due to the stiffening of the mix. As the water becomes more and more mixed with the dry materials, the curve begins to drop after a few seconds and finally becomes horizontal. It is safe to assume that the concrete is thoroughly mixed when the curve straightens out horizontally.

TEST DATA

It is not the purpose of this paper to discuss all phases of the problem of workability. The authors have, however, given some study to two angles of this problem and are presenting the results of two series of tests, in the form of the data and curves of Figs. 3 and 4.

Study of Workability as Influenced by Variations in Proportions (Fig. 3):

In our first study the following conditions apply:

1. The cement, sand and gravel are from the same source for each mix.

2. The mixes, which were the variables in this group, were designed using as a basis Abram's Curve "A." In order to bring these mixes to a consistency that would give a slump of from 3 to 4 in., it was necessary to modify them somewhat, as shown in the figure.

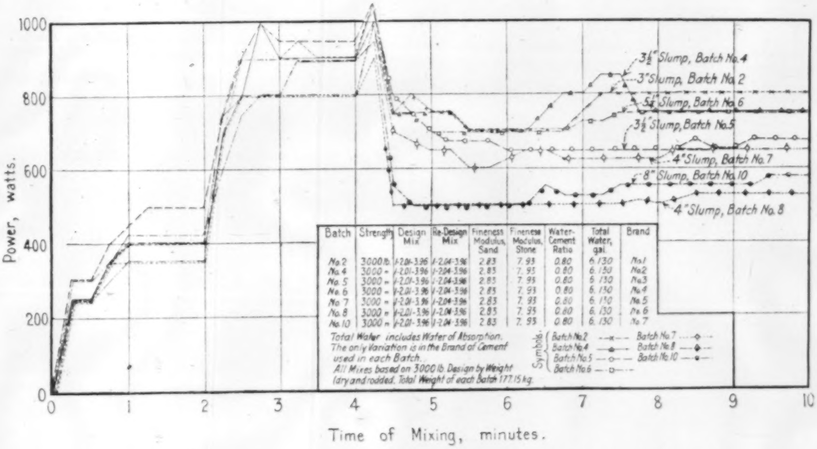
3. In each case a calculated amount of water was added to compensate for that absorbed by the materials.

4. The procedure carried out for adding the various materials was exactly the same in each case.

5. Readings were taken at 15-second intervals.

The batches show quite consistent results up to 8 minutes and 45 seconds. As the mixes become richer in cement, the wattage increases. This is to be expected, for the richer the mix (slump remaining constant) the more "sticky" the concrete becomes.

An interesting study was made using celite as an admixture. The results have been plotted and designated as batch 9 in Fig. 3. The 3000-lb. mix was used with the addition of 3 lb. of celite per bag of cement. No additional water was added at first, but later increments of water were introduced until the total amount had been added. While the slump increased it would appear that the wattage increased somewhat, probably due to the long mixing and consequent absorption of water by the cement and celite. The method of test opens up some interesting possibilities for study of the effect of admixtures on concrete.



Batch	Strength	Design Mix	Re-Design Mix	Fineness Modulus, Sand	Fineness Modulus, Stone	Water-Cement Ratio	Total Water, gal.	Brand
No. 2	3000 lb.	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 1
No. 4	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.150	No. 2
No. 5	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 3
No. 6	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 4
No. 7	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 5
No. 8	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 6
No. 10	3000 "	1-2.01-3.96	1-2.04-3.96	2.83	7.93	0.80	6.130	No. 7

Total Water includes Water of Absorption.
The only Variation is in the Brand of Cement used in each Batch.
All Mixes based on 3000 lb. Design by Weight (dry and rodded. Total Weight of each Batch 177.15 kg.

FIG. 4.—Results of Tests on Seven Batches of Concrete of Different Cements.

Study of Effect of Cement on Workability (Fig. 4):

The given conditions under which work was carried out in the next series of tests were somewhat different from those used in the foregoing one. In this case the procedure was as follows:

1. The sand and gravel were the same in each mix.

2. In each case the kind of cement was the variable. The cements were furnished by the manufacturers and were of different plasticities.

3. The mixes were the same as represented by batch 2, Fig. 3.

Three manufacturers of cement furnished two samples each, from different mills. In each case, one sample was known to be more plastic than the other. The results plotted on the curves followed out predictions with the exception of batch 6, for which a slump of 5¼ in. was obtained. When the concrete in this batch was thoroughly mixed it was very adhesive. In fact, this quality was more pronounced in this case than in any other.

The methods used in the mixes for which the results are submitted in this paper have been used with other sands and gravels. While different absolute values were obtained the relationship of the various cements to wattage is about the same.

SUMMARY

In respect to the data included in this paper, the following facts were noted:

1. Power consumption increases with the richer concrete mixtures.
2. The time required uniformly to mix concrete may be determined by the power curve.
3. The rate of absorption of water by the cement particles varies greatly with different brands of cement; the greater the hydration the greater the power consumption and *vice versa*.
4. The investigation opens a way for productive research on various types of admixtures.

[For Discussion on Workability of Concrete, see page 521.—ED.]

THE MEASUREMENT OF WORKABILITY OF CONCRETE

By George A. Smith[1]

Synopsis

The paper describes an improved penetration apparatus for measuring workability of concrete and shows that indices obtained are commensurate with observations made in the field and with the judgment of experienced investigators.

Data are presented for concretes in which some of the elements, which are recognized as having decided effects on the workability, such as cement content, fineness of sand, etc., were varied. Under these conditions the apparatus gives an index which is representative of observed workability.

Variations of results obtained from single tests, though occasionally larger than would be desired, are not excessive. Results obtained demonstrate that the average of the fifteen tests made gives a reliable indication of the workability.

Introduction

That workability is being recognized as an important factor in the manufacture and use of concrete was evidenced by the unusual interest shown at the recent meeting of the American Concrete Institute. The several papers at the symposium dealing with the effect of workability in relation to construction only emphasized the need for numerically evaluating this important quality of concrete. In a paper[2] by Mr. George Conahey and the author, the definition of workability was given as that property which is indicated by the effort required to place the concrete in order to obtain a uniform and homogeneous finished product. There were described a number of methods for measuring workability which had been tried with varying success. And it was pointed out that the most promising device was one in which was measured the energy required to drive a system of three rods into a mass of fresh concrete.

Not infrequently it is pointed out that variations in the constituents which enter into the making of concrete play a material part in causing changes in the workability of the final product. The difference between the workability of a rich mix and a lean mix is readily discernible and is generally recognized. In the designing and proportioning of concrete the effect of the grading and character of the aggregates is given considerable attention, not only from the standpoint of securing the desired strength but also with the view of securing a workable mix. If in the manufacture of concrete the mix is stiff and difficult to handle, a small amount of water will remedy the situation by making the concrete more workable. In their early work, Messrs. Pearson and Hitchcock pointed out that various powdered materials had beneficial effects in promoting workability. The advantages to be had from the use of these materials is rather widely recognized and appreciated among the craft.

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[1] Engineer, Celite Products Co., Lompoc, Calif.

[2] George A. Smith and George Conahey, "Some Methods of Measuring Workability of Concrete." *Proceedings*, Am. Concrete Inst., Vol. 24 (1928).

As the difference in the workability of two concretes is a matter of opinion of those who come in intimate contact with it from the time of discharging the mixer to the time the concrete is finally in place, the distinguishing of differences in workability within narrow limits is a difficult task unless some device can be used which will point out relatively small differences. Recent studies with the penetration apparatus seem to indicate that the device will point out distinctive differences in the workability and give a reasonably reliable index of that property of a concrete.

It is the purpose in this paper to describe the apparatus in detail and to present new data which prove conclusively that the apparatus gives true indices of workability compared with observations of that property in the field. The subject is necessarily a very broad one and a great many tests would be necessary to give the complete story of the effects that all conditions and changes would have upon it. In the study undertaken, tests were made in which were considered some of the factors recognized as having an important bearing on this property.

MATERIALS AND PROPORTIONS USED IN TESTS

The basic proportion to which all tests are referred was a nominal mix, 1:2:4 by volume, equivalent to one part of cement to five of mixed aggregates. In proportioning the batches, all materials were weighed to give proper volume proportions. For the study of the effect of varying the cement, the quantity was increased and decreased 20 and 33½ per cent from the basic mix. This gave, for the range considered, nominal mixes between 1:1½:3 and 1:3:6.

During the carrying out of the program it was necessary to use two lots of cement to complete the tests. The same brand of cement was used, however, and tests indicated that the two lots were very much the same. Each lot was carefully mixed and stored in tight cans until used.

The powdered materials used for increasing workability were celite (Plankton marine diatomaceous silica), another type of diatomaceous silica, hydrated lime and volcanic ash. The proportions in which the materials were used were 3 and 5 lb. per sack of cement for the diatomaceous silicas, 6 and 10 lb. for volcanic ash, and 9 and 15 lb. for hydrated lime.

In preparing the aggregates the greatest care was exercised to insure uniform mixes. The sand, which was a washed river sand, was separated on a No. 16 sieve. The coarse fraction was divided into small lots by means of a Jones sampler and later cut down by the same means to about the quantity specified for each. The fine sand was moistened with about 7 per cent of water to eliminate segregation, thoroughly mixed and sacked. Each sack contained enough fine sand for four batches. This was allowed to dry out in the sacks and by spreading on large sheets of paper until there was less than 0.1 per cent of free moisture. Each sack of fine sand was cut into four parts by means of the sampler. For the entire investigation the two sands were combined to give a resulting fineness modulus of 2.75, except for those tests in which it was desired to show the variation in workability due to using fine sand and coarse sand. From eight analyses of each sand it was found that the fineness modulus for the fine varied from 1.91 to 1.93 and that of the coarse from 3.93 to 4.04.

The coarse aggregate was river gravel, which was washed, dried, and separated into three sizes, No. 4 to $\frac{3}{8}$ in., $\frac{3}{8}$ in. to $\frac{1}{2}$ in., and $\frac{1}{2}$ in. to $\frac{3}{4}$ in. Each batch of concrete contained these three sizes combined in the ratios, 22 per cent fine, 33 per cent intermediate and 45 per cent coarse, for all tests except those in which the gradation of the coarse aggregate was varied. For these tests the gravel was graded No. 4 to $\frac{3}{8}$ in., $\frac{3}{8}$ in. to $\frac{3}{4}$ in., and No. 4 to 1 in.

The crushed rock used was a conglomerate from Honda, Calif. It was broken up in a small crusher, screened and recombined to give the same gradation as was used for the No. 4 to $\frac{3}{4}$-in. gravel aggregate.

Each batch of materials sufficient to make a little more than 0.40 cu. ft. of concrete was mixed dry and then for two minutes after adding the water. All procedure preparatory to making the workability tests was in accordance with the A.S.T.M. Standard Methods of Making Compression Tests of Concrete (C 39 – 27)[1] except in the case of making the consistency determination. This was done by means of a flow-table adjusted to give a drop of $\frac{1}{8}$ in. instead of $\frac{1}{2}$ in., as it has been found that this gives greater sensitiveness, a broader range and greater accuracy to the test.[1] The flows indicated are the percentage increase in diameter of the sample.

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[1] 1927 Book of A.S.T.M. Standards, Part II, p. 112.

APPARATUS AND PROCEDURE

The workability tests were made with the improved penetration apparatus shown in Fig. 1. In Fig. 2 is shown the assembly of the apparatus.[2] The device is based on the early apparatus used by Messrs. Pearson and Hitchcock[1] but with certain changes intended to improve its functioning and the reliability of the results. The container for the fresh concrete is twice as large in cross-section and three hollow $\frac{1}{2}$-in. rods are used instead of the single $\frac{3}{4}$-in. solid rod.

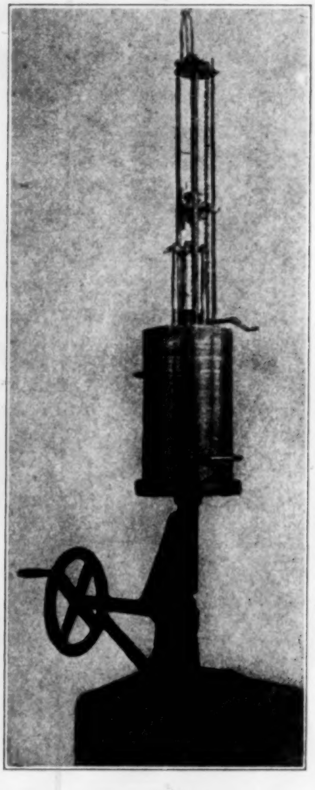

FIG. 1.—Improved Penetration Apparatus.

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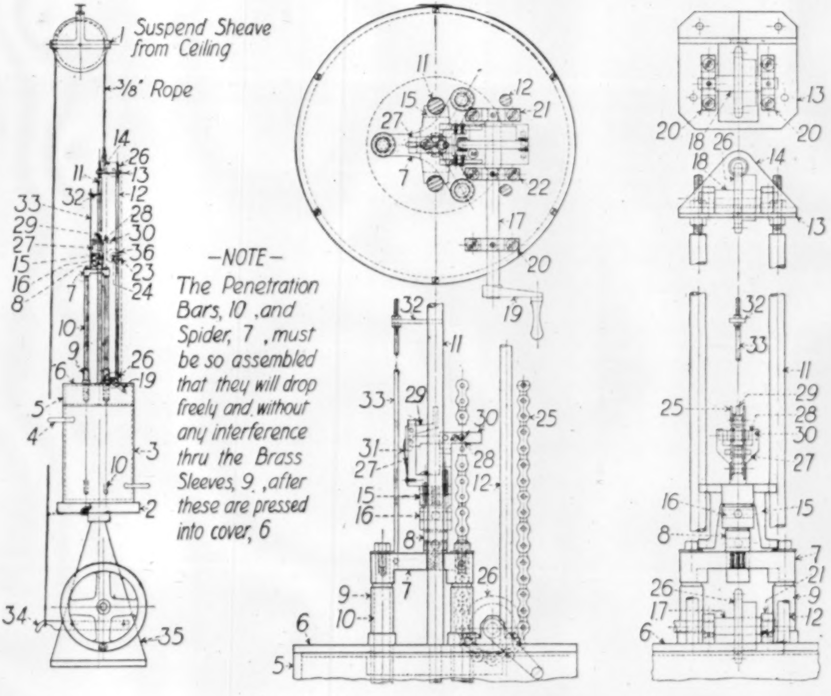
[1] Pearson and Hitchcock, "Workability of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 281 (1923).

[2] Drawings showing the details of construction of the apparatus may be secured upon application to the author.

In the early tests,[1] the rod was allowed to settle through the concrete under its own weight during the jolting of the concrete. This produced a consolidating of the concrete which penalized the leaner and harsh-working mixes. To overcome this, a hammer dropping from a fixed height above the rods is used to cause the penetration. The rods and driving mechanism are mounted on a removable head which fits in the top of the container. Three sleeves in the head act as guides for the rods which are rigidly connected by a spider. In the center of the spider is an anvil for receiving the blow from the hammer. The hammer is operated by means of a sprocket chain with pins which come in contact with a yoke. The tripping device consists of a mousetrap arrangement attached to the hammer. After being elevated to the desired height the mousetrap trips the yoke and the hammer falls freely. Upon striking the anvil the trap automatically latches for the next elevating of the hammer.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–NOTE–
The Penetration Bars, 10, and Spider, 7, must be so assembled that they will drop freely and without any interference thru the Brass Sleeves, 9, after these are pressed into cover, 6

FIG. 2.—Assembly of Apparatus.

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[1] Pearson and Hitchcock, "Workability of Concrete," *Proceedings*, Am. Soc. Testing Mats, Vol. 23, Part II, p. 281 (1923).

In making the tests a batch of concrete is mixed by hand to the desired consistency and placed uniformly in the container to within $\frac{1}{4}$ in. of the top of the mold. The concrete thus deposited is then jolted, dropping it $\frac{1}{8}$ in. thirty times by means of a jigging table.

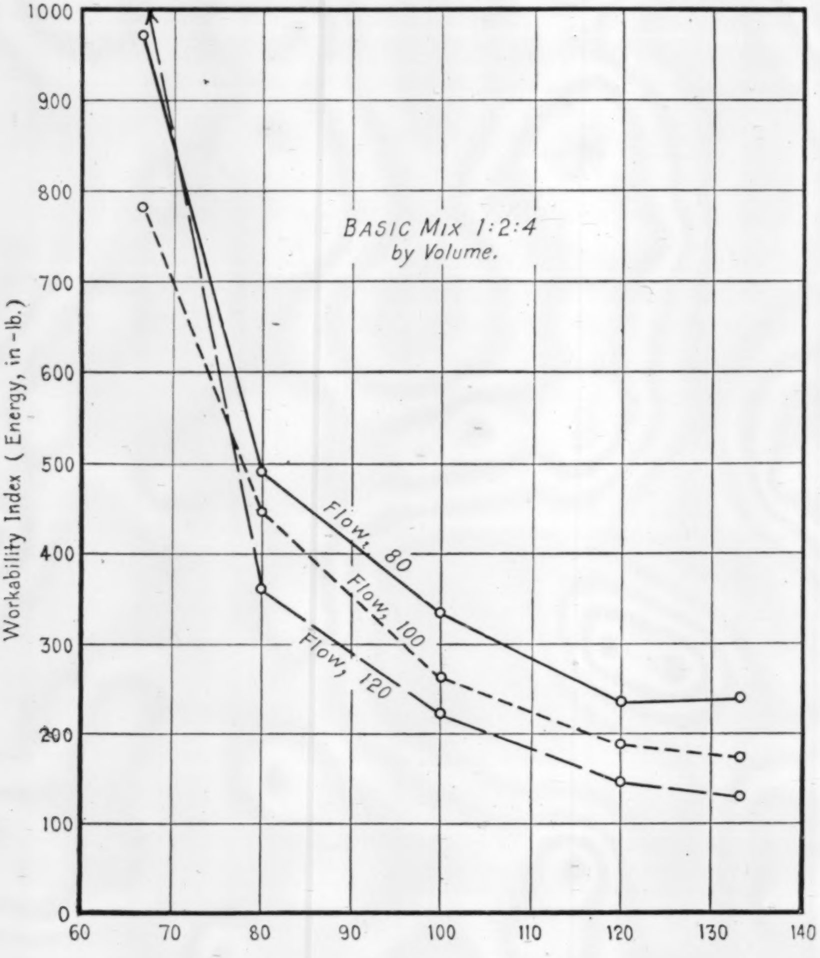

FIG. 3.—Variations in Workability Due to Varying the Proportion of Cement.

The driving mechanism is then placed on top of the mold, raising the rods so that they do not come in contact with the concrete. The rods are then lowered to the zero mark and allowed to drop freely, penetrating the concrete under their weight and that of the hammer. Further penetration to the desired depth, 11 in., is accomplished by successive blows of the hammer. It should be pointed out that there is no manipulating of the concrete subsequent to its being placed in the mold, other than the jolting, and particular care is exercised at all times to insure a uniform distribution of the material.

Three workability tests were made on each of five batches of concrete for each condition of the investigation. With but a few minor exceptions no two batches for the same condition were made on the same day.

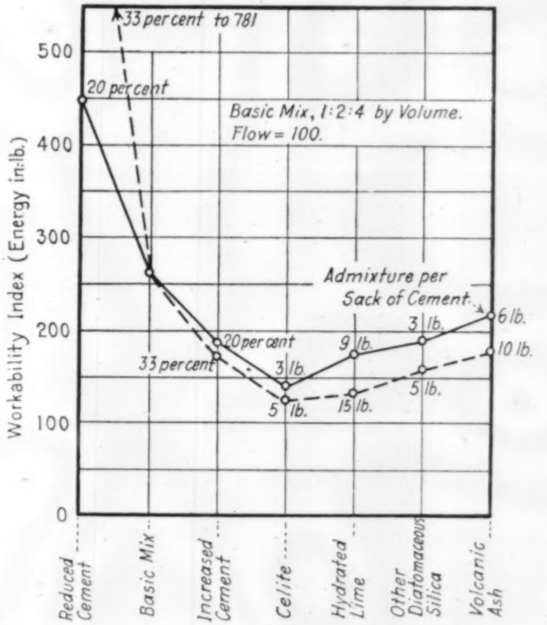

FIG. 4.—Summary Showing the Effect on Workability of Various Materials Added to the Mix.

The workability indices obtained are based on the average number of blows required to give an 11-in. penetration. The indices were computed by the following equation:

$$W = p(R_w + H_w) + nh\,H_u$$

where p = depth of penetration in inches,
R_w = weight of rods in pounds,
H_w = weight of hammer in pounds,
n = number of blows, and
h = drop of hammer in inches.

The index thus obtained represents the energy expended in driving the rods through the concrete. Since resistance to penetration decreases as the workability increases, the workability of two concretes will be inversely proportional to the indices obtained.

TEST DATA

Effect of Varying Proportion of Cement on Workability:

The advantage of using richer mixes is universally recognized and extra cement is often used for no other reason than to improve the workability. In Fig. 3 are shown the workability indices for concrete containing varying amounts of cement and gaged to different consistencies. The harsher working mix, due to reducing the cement content 20 per cent below the basic content, was about half as workable as the basic mix at the same consistency. The mix containing two-thirds as much cement as the basic mix was one-third as workable, and one-half as workable as the mix with the 20 per cent reduction in cement. The indices obtained when the cement was increased 20 and 33⅓ per cent indicated improvements of 39 and 52 per cent, respectively.

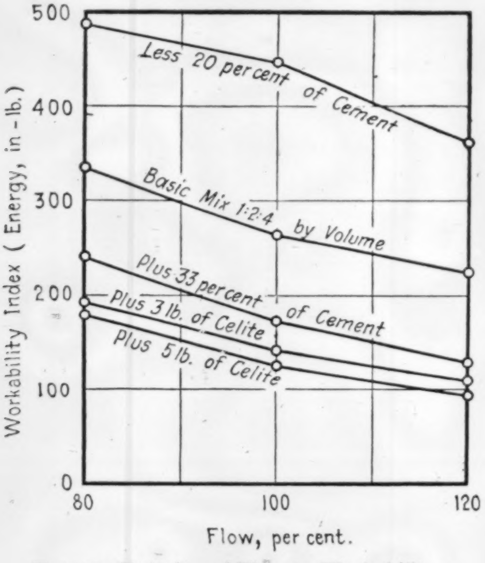

FIG. 5.—Relation of Flow to Workability.

Relation of Flow to Workability:

The effect of variations in cement content with and without admixtures for flows of 80, 100 and 120 are shown in Figs. 3, 5 and 8. The curves shown in Fig. 5 are characteristic of values plotted in the other figures. The workability index for any one of the concretes at a flow of 80 is higher than that for a flow of 100 or 120. At a flow of 80, the mass was stiff and about as dry as could be used for most reinforced construction. Concrete at a flow of 100 was at about the optimum consistency. That at 120 flow was rather wet and in some instances there was a tendency for the excess water to run away from the mix. As the flow was increased the concrete became easier to handle and was judged more workable. The variation in the indices parallels the variation in the observed workability of the concrete. An improved observed workability is clearly indicated by a lower index. It has been pointed out by Talbot and Richart[1] that within a certain range an increase in water is accompanied by an improvement in workability, but beyond this range the excess water does not improve the workability and tends to increase segregation. Also when excess water is used it will not remain uniformly distributed. The change in the region of 120 flow was small and as shown by the indices the workability was approaching a maximum beyond which there would be no improvement. This condition was clearly pointed out by indices obtained in the original tests with the single-rod apparatus.[2]

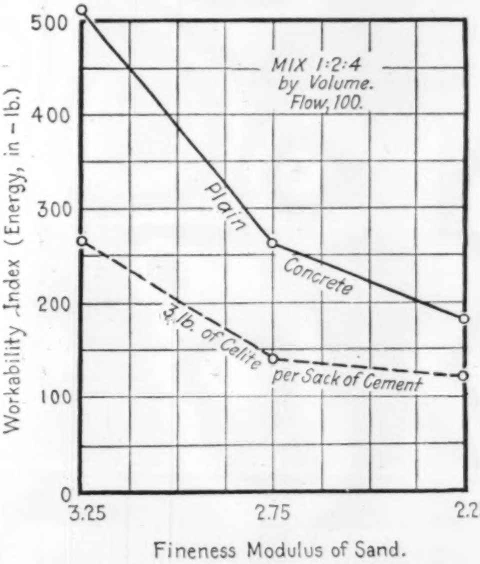

FIG. 6.—Effect of Gradation of Fine Aggregate on Workability.

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[1] University of Illinois *Bulletin No. 137*, p. 93.

[2] George A. Smith and George Conahey, "Some Methods of Measuring Workability of Concrete," Fig. 8, *Proceedings*, Am. Concrete Inst., Vol. 24 (1928).

With the mixes in which the cement content was reduced $33\frac{1}{3}$ per cent it was found that the rods could not in all cases be driven to the full 11-in. penetration established as standard for this instrument. This is probably due to compacting of the ingredients in the bottom of the container due to segregation. Hence, a large number of blows correctly indicates poor workability, as required by the definition of that term. Under test conditions in which the rods did not move appreciably under a number of blows and a 10-in. penetration was not reached, the test was discontinued at 250 blows. For this particular mix, the reduction in cement was so large that the concrete had more the semblance of a granular mass than of a plastic cement.

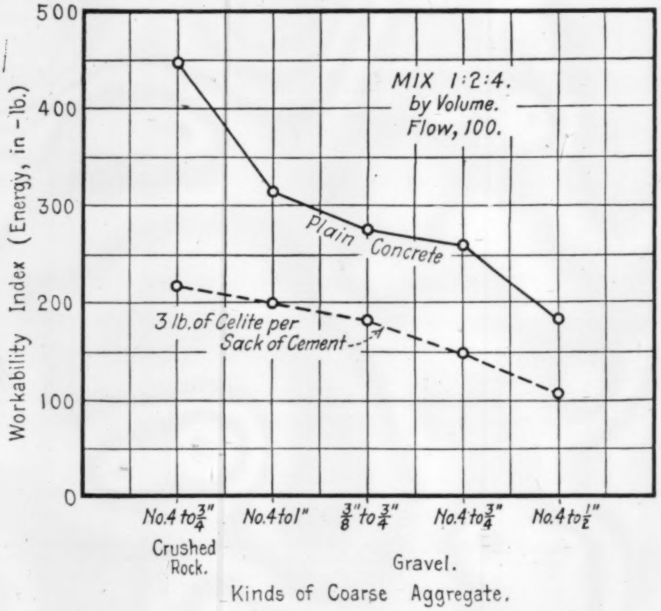

FIG. 7.—Effect of Coarse Aggregate on Workability.

Effect of Gradation of Aggregate on Workability:

Although sand grading specifications on many jobs are quite rigid, it is probably not generally realized what value a fine sand will have in aiding workability or how harsh a mix will result from the use of a coarse sand. In Fig. 6 are the indices for the workabilities of concretes made with sands having fineness moduli of 3.25, 2.75 and 2.25. It was observed that sand with a fineness modulus of 3.25 gave a concrete which was harsh and decidedly less workable than that in which the 2.75 sand was used. With the sand having the 2.25 fineness modulus, however, quite the opposite was observed, the mix being far more workable than those with the other sands. Parallel observations were pointed out by Pearson and Hitchcock.[1] They considered that a concrete made with sand having a fineness modulus of 3.21 was very harsh and exceedingly unworkable, and that concrete made with fine sand was an unusually workable mix. Messrs. Talbot and Richart,[2] in considering the elements affecting workability, point out that a "fine sand acts somewhat as does cement." The high index for the coarse sand and the low index for the fine sand, shown in Fig. 6, are in agreement with the observed workability of the respective concretes.

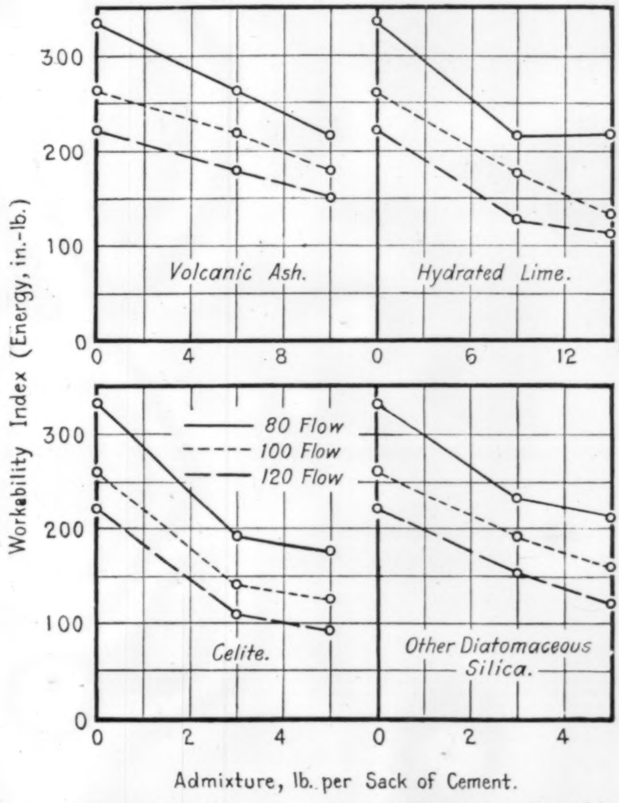

FIG. 8.—Effect of Powdered Admixtures in Improving Workability.

Although the apparatus is particularly intended for use with aggregate having a maximum size of $\frac{3}{4}$ in., the program of tests for variations in the gradation of the coarse aggregate included 1-in. gravel. There was also included one condition using crushed rock graded No. 4 to $\frac{3}{4}$ in. Results obtained with the different coarse aggregates are shown in Fig. 7. The workability index for the various concretes increases with the coarseness of the aggregate used.

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[1] Pearson and Hitchcock, "Workability of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, pp. 291–292 (1923).

[2] University of Illinois *Bulletin No. 137*, p. 92.

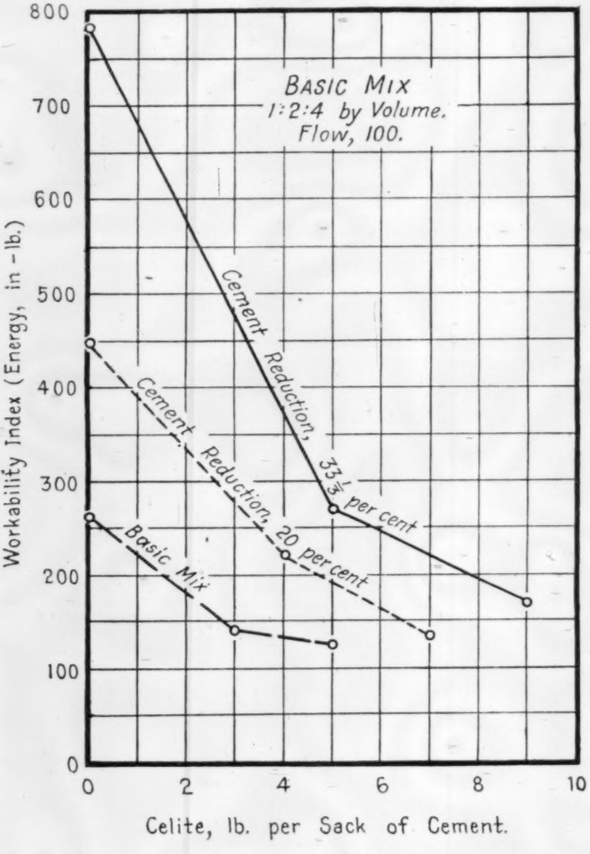

FIG. 9.—Influence of Diatomaceous Silica in the Leaner Mixes.

In the case of the crushed rock aggregate an index 74 per cent higher than that for the similarly graded gravel was secured.

Small aggregate, such as pea gravel, is recognized as being conducive to workability. Concrete with such small coarse aggregate approaches the limits of a lean mortar and would naturally be expected to have high workability. Increasing the size of the aggregate would naturally be expected to decrease the workability, but to what extent is very difficult to determine by observation. The decrease in workability due to this factor will naturally be variable and depend upon the conditions under which the concrete is to be placed.

The use of crushed rock and coarse aggregate results in a concrete somewhat less workable than that in which gravel is used. This is due to the fragmentary shape of the particles, which prevents their being moved or displaced in the mix as easily as the smoother and more rounded pebbles The effect of this on the workability resulted in a higher index than was obtained with gravel aggregate of the same grading.

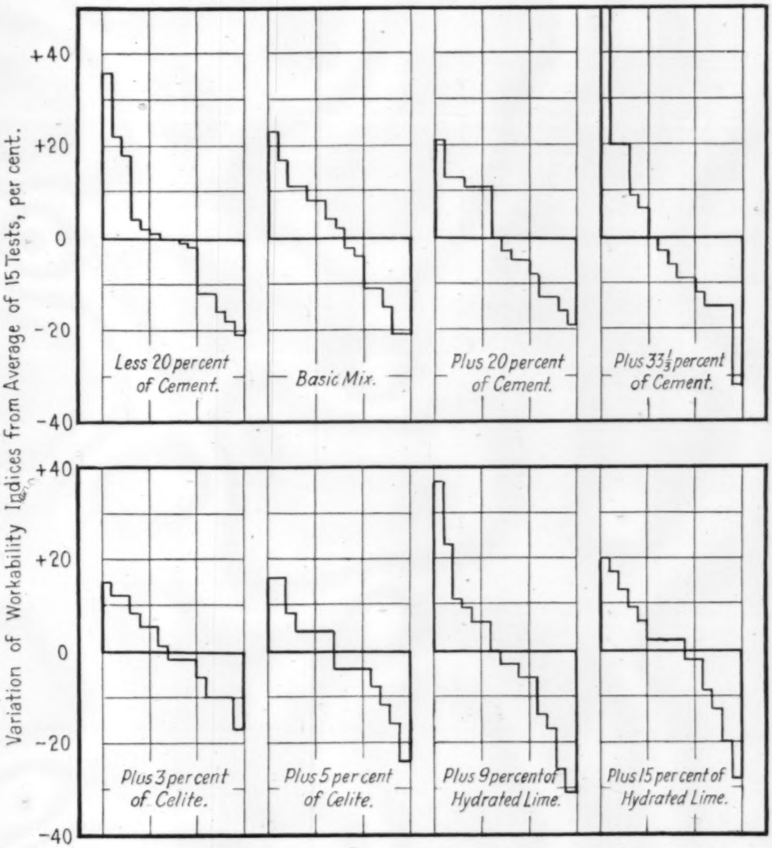

FIG. 10.—Variation of Results of Single Determinations from the Average of Fifteen Tests. Basic Mix, 1:2:4 by Volume; Flow 100.

Effect of Admixtures on Workability:

For a number of years the effect of hydrated lime in promoting workability of concrete has been recognized and since the work of Messrs. Pearson and Hitchcock, other powdered materials have been used to impart to concrete that quality which is essential to the securing of a satisfactory structure. Figure 8 shows the results obtained with the several materials considered in these tests. Figures 6, 7 and 9 also show results of tests where celite was added to mixes in which the cement, or the sand, or the gravel was varied. In all the mixes containing admixtures there was a marked increase in the plasticity and the effect observed was quite similar to that experienced when cement was added. However, the workability of the various concretes as judged during the making of the tests was noticeably different with some materials than it was with others. In general, the admixtures increased the plasticity of the mix and prevented to a marked extent a segregation of the ingredients, which was quite pronounced in the harsher working mixes. The addition of celite (Fig. 9) in the leaner mixes was accompanied by a marked improvement in the plasticity and a decided lowering of the workability index.

Where cement is used to improve workability, that amount not required in developing the proper strength may be considered as an admixture for the sake of improving the ease of handling and placing. Figure 4 shows the relative effect of varying the cement and additions of different amounts of the admixtures.

In connection with his work at the Bureau of Standards, Mr. George Conahey has made tests with the penetration apparatus and has kindly offered the following data:

Admixture		Water-Cement		Workability
Kind	Amount per Sack of Cement, lb.	Ratio, $\frac{W}{C}$	Flow	Index
None	0	1.120	91	136
Cement	31	0.882	91	67
Celite	1½	1.160	93	100
Celite	3	1.210	93	69

In these tests a 1:2:4 mix of a commercial cement on the Washington market and Potomac River sand and gravel was used.

The following data are also furnished by Mr. Conahey:

Mix	Aggregate	Water-Cement Ratio, $\frac{W}{C}$	Flow	Workability Index
1:2:4	Potomac River Gravel	1.12	91	136
1:2½:4	Potomac River Gravel	1.23	89	118
1:2½:4	Blast-Furnace Slag	1.43	88	367
1:2½:4	Trap Rock	1.43	87	377

The coarse aggregates for Mr. Conahey's tests were graded 50 per cent of No. 4 to $\frac{3}{8}$ in. and 50 per cent of $\frac{3}{8}$ in. to $\frac{3}{4}$ in.

Although there are different indices obtained for similar mixes by Mr. Conahey and the author, it should be borne in mind that the type. character and gradation of the coarse aggregate were different. These factors would affect the workability. The facts that are significant are that corresponding changes in the proportions or in the materials, produced like changes in the workability indices.

Uniformity of Results:

During the making of these and other tests it was observed that indices obtained for individual tests with an occasional mix were consistently somewhat higher than those for other companion mixes. It was not until the tests were made with the sands having widely different fineness moduli that a possible explanation for this condition could be made. With the maximum variation possible in the fineness modulus due to combining the sands as analyzed, there would be a variation in the workability index of about ±15 for the basic mix at a flow of 100.

An occasional high or low value can easily be attributed to variations in the sample as it is arranged in the mold and to intrinsic conditions within the mix.

In Fig. 10 are shown the percentage variations from the average of fifteen tests for eight conditions. It will be observed that there is an occasional high or low value. In the fifteen tests, however, there is usually a low value to offset the high one. Generally, the greater number of results are within 20 per cent of the mean value; and of the results here considered, over 50 per cent were within 10 per cent of the average.

During the making of those tests in which the coarse aggregate was varied, the tests with the No. 4 to $\frac{3}{4}$-in. gravel were repeated, because of the change in the cement. In Fig. 6, for sand having a fineness modulus of 2.75 the index for plain concrete is 261, and for the concrete in which celite was used the index was 140. For similar conditions but using the second lot of cement, the indices were 258 and 148 (see Fig. 7).

While the variations of single tests from the mean are sometimes larger than would be desired, still they are no larger than variations observed in other concrete tests in which there is no known variable that should affect the results. Tests of field specimens have shown a variation in the compressive strength of 125 per cent from the minimum, sometimes from the same batch of concrete.[1]

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[1] G. W. Hutchinson, "Uniformity of Field Concrete," *Concrete*, October, 1925.

CONCLUSIONS

From the changes in the workability indices, as single constituents of the mix were varied, it appears that the improved penetration apparatus is capable of demonstrating in a consistent manner changes in the workability due to these variations. The workability indices obtained pointed out clearly variations in the workability of the concrete as they are observed in the field. Proportionate decreases in the indices due to increasing the cement content, or the fineness of the sand, decreasing the maximum size of the coarse aggregate, or changing the flow, are commensurate with recognized improvements in the workability of concrete.

Although results of single tests occasionally vary somewhat more from the average value than is desirable in other tests, yet it is believed that the average of fifteen tests gives a true and reliable index of the workability for conditions under which the tests were made.

The indices obtained from the tests reported herein are consistent with field observations and with the judgment and observations of experienced investigators. Within the range of plastic mixes the apparatus functions to give a true index of the workability. This method of measuring workability makes it possible to represent that property of the concrete by an index which will be comparable with indices obtained under other conditions. The test is, therefore, recommended as a satisfactory method for evaluating this particular and very important property of concrete, workability.

DISCUSSION ON WORKABILITY OF CONCRETE

Mr. Conahey.

MR. GEORGE CONAHEY.[1]—I should like to compliment Mr. Purrington on the initiative shown by his efforts in trying to measure this property of workability in which so many of us are interested. His apparatus does not show very great differences between the 2000-lb. concrete, the 3000-lb. concrete and the 4000-lb. concrete. These differences would, no doubt, be made greater if he would change the shape of the blades in the mixer, so that the longer one would plow the concrete from the edge of the mixer toward the center and the shorter blade would plow the concrete from the center toward the edge, thus giving greater resistance to shear. The large element of friction now in this apparatus could be reduced by redesigning the apparatus if it would measure the differences in workability of the concrete mixtures. Either the mixer proposed by Mr. Chapman or the mixer of Mr. Purrington, if they could be made to measure this property, would be ideal methods of measuring workability, for then the concrete could be mixed and a measure of the workability made, the concrete then dropped from the mixer for making a slump or flow test or any other measurement and the test specimens could be made without any lost motion.

Mr. Herschel.

MR. W. H. HERSCHEL.[2]—Mr. Conahey has said some kind words for the paper by Mr. Purrington, but I do not think he has brought out the theoretical advantages of this type of mixer. The method of penetration is fundamentally defective, I believe, in ignoring the fact that the resistance depends upon the speed. In measuring viscosity, the speed of flow is the important thing. I believe that speed is equally important when it comes to plastic materials; it is not a question of the number of foot-pounds that are used, because the number of foot-pounds would vary with the speed at which the work was accomplished, but it is a question of power. Now, breaking away from the penetration test, Mr. Purrington uses power as a measure of workability, and therein I believe he is correct.

As pointed out by Mr. Conahey, his method, being a pioneer one, may be capable of being improved. For instance, the very large friction of 250 watts that is required to run the instrument without any charge is between a third and a fourth of the entire power, and therefore this seems rather large and doubtless could be cut down. Very

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[1] Research Engineer, The Celite Products Co., New York City.

[2] Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

Mr. Herschel. likely there is a wedging between the paddles on the bottom of the container that accounts for a considerable portion of the friction when the charge is in place. As Mr. Purrington says, it is not expected by this one test to solve all the difficulties, and in the paper by Mr. Smith it is said that his method applies only to materials within the plastic range, which leaves it undetermined how one is to determine whether the material in question is or is not within the plastic range. It is attempting too much to measure workability by a single test. Plastic materials, unlike viscous liquids, do not have a rate of flow which is in proportion to the force applied; therefore a test at a single rate of flow does not give complete information. Mr. Purrington does not give a very complete interpretation of his results. It cannot be said that the material which requires the lowest power is necessarily the best material. Pure water would require the least power but would not be good concrete. What we need to know is that the resistance to flow, at very low rates of flow, is sufficient to prevent segregation; then, if the resistance at low rates of flow is adequate, then the less the resistance at high rates of flow, the better, because the material is more easily moved. What the workman who handles the end of the shovel wants to know is not the ultimate power required to move the material, but the rate of work or horsepower that he is called upon to expend.

Mr. Conahey. MR. CONAHEY.—In answer to Mr. Herschel's criticisms, we should like to say before we attempted to improve the penetration apparatus of Pearson and Hitchcock, we attempted to measure the plastic properties of neat cement. We hoped to develop an apparatus that would measure the plastic properties of the neat cement paste, change the apparatus so that it would measure the plastic properties of the cement mortars and then to enlarge the apparatus and obtain the same properties for concrete. Much to our surprise and disappointment, this did not work out. We found that a mixture of cement and water was a plastic material as defined by the Technical Committee on Plasticity, Consistency, etc, of Committee E-1 on Methods of Testing.[1] That is, it had a rate of flow to pressure relation which when plotted was a straight line relation not through the origin. After we learned this, we attempted to measure the plastic properties of mortar, and we found that so long as there was an excess of cement in the mortar it acted as a plastic material, but when there was an excess of sand, as in a lean mix, it resembled a granular material.

There are only a few things that we know about the properties of granular materials. Mr. Osborne Reynolds of England made a

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 621 (1928).

Mr. Conahey.

study of these properties and published his results in the *Philosophical Magazine* in 1888. He states that a granular material exhibits that property which he names dilatancy. That is, it will expand if the shape of its mass is changed, when it is in the closest packed condition. This property may not be readily understood, but as an illustration of it: If you go out on the beach where the waves have come up and have inundated the sand and you put your foot down and make an impression in that sand, you will see the sand rise up around your foot to a much greater degree than is necessary to allow for the small indentation made by your foot. As inundated, granular material will assume the closest packed condition. You all know that from the use of the inundator in sand measurement. Another property is that a granular material will bulk or swell when dampened. You know that from your measurement of sand also. If you will mix plastic materials and granular materials in such proportions that on one end of the scale you have a plastic material and at the other end of the scale you have a granular material, the methods used to measure the plastic properties of the plastic material will not give satisfactory results when used to measure the properties of a granular material, for this material is not plastic. A point of interest along this line was brought out by the work of Messrs. Bates and Dwyer of the U. S. Bureau of Standards and reported to the American Concrete Institute last February.[1] Their results showed that when they obtained the flow-pressure relations to measure the plastic properties of neat cement paste, the property of the material termed mobility, that is, the angle at which the curve approached the horizontal axis, was very nearly the same for all brands of cement. This would indicate that if we can obtain a method of measuring what is known as the yield value of a material, we would have a measure of one of the most important factors which determine the workability of a concrete or mortar.

The penetration apparatus may not be the final method selected for measuring the workability of concrete, but it is the best one that has been developed thus far. Mr. Purrington has shown us some very interesting results with a mixer. Mr. Cloyd M. Chapman has done some work along the same lines with another type of mixer. Since our report was made to the American Concrete Institute[2] we have completed more work with the deforming cylinder apparatus and some very interesting results have been obtained. It may be that any one of these four pieces of equipment, that is, the penetration

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[1] P. H. Bates and J. R. Dwyer, "Cement as a Factor in the Workability of Concrete," *Proceedings*, Am. Concrete Inst., Vol. XXIV, p. 43 (1928).

[2] G. A. Smith and George Conahey, "A Study of Some Methods of Measuring Workability of Concrete," *Proceedings*, Am. Concrete Inst., Vol. XXIV, p. 24 (1928).

Mr. Conahey. apparatus, the deforming cylinder apparatus, Mr. Purrington's mixer or Mr. Chapman's mixer might work out to be the best apparatus, but that is something that a newly appointed Sub-Committee on Workability of Committee C-9 on Concrete and Concrete Aggregates should investigate.

Mr. Purrington. MR. WALLACE F. PURRINGTON.[1]—One of the advantages in using this machine which we suggest, is that it will use any type of aggregate. We have now worked on aggregates up to 2 in. in size. In answer to the statement made by Mr. Herschel in regard to friction, I think this factor is pretty well minimized, due to the way the machine is built. It would take some power to even run the motor without any load whatever on it. As far as the paddles are concerned, they do not touch either the sides or the bottom of the tub. We called the wattage used with the machine running without load our machine factor, which is very constant.

Mr. Dwyer. MR. J. R. DWYER[2] (*by letter*).—The paper by Mr. G. A. Smith contains matter of great interest in the study of the workability of concrete, but without the presentation of the necessary details to support some of its conclusions it suggests rather than answers questions. Briefly, the following points are of sufficient importance to warrant further data:

1. It is stated a number of times that the workability indices are commensurate with field observations, and yet there is not even a summary of the methods and results used in arriving at this conclusion.

2. Figure 3 shows that the flow does not vary directly with the workability index, but that the ratio varies widely. The flow table is used as the means for determining the water content of the various mixes and it is stated that "concrete at a flow of 100 was at about the optimum consistency." Thus the flow table appears to show, by the same value, the optimum consistency for widely varying mixes, and yet the penetration device indicates workability indices varying in the ratio of 1 to 4 for this consistency, and 1 to 8 for flow of 120. Long use of the flow table makes one reluctant to admit that, even with its deficiencies, its indications are so poorly representative of the workability of concrete. When flows could not be secured for the leaner mixes without segregation or loss of water, such fact should be taken into account in the curves of Fig. 3.

3. In Fig. 5, for a flow of 100, there are approximately 10 to 25 points variation in the workability indices of three mixes from an average index of about 150, or only 10 to 15 per cent variation, and

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[1] Materials Engineer, New Hampshire Highway Department, Concord, N. H.

[2] Associate Engineer Physicist, U. S. Bureau of Standards, Washington, D. C.

Mr. Dwyer. yet in Fig. 10 the range of variation of results of the penetration tests of one mix is 80 per cent, and in no case is it less than about 30 per cent. The strength variation of 125 per cent is hardly sufficiently representative of desirable practice to warrant its use as a basis for a favorable comparison of a laboratory device, when the difference in the average effects of certain admixtures studied may be small compared to the range of the individual results. Does a second series of 15 tests indicate that the range of variations is not excessive?

4. What is the means of determining the limits of the range of plastic mixes within which the penetration apparatus gives a true index of the workability? Is the 1:3:6 mix, at a flow of 120, considered within this range?

Mr. Smith. MR. G. A. SMITH[1] (*author's closure by letter*).—Mr. Dwyer's first observation relative to the methods and results used in arriving at the conclusion is answered by the last paragraph of the paper where it is stated that the results are consistent with field observations and with the judgment and observations of experienced investigators. Differences in the workability of mixes having different cement contents, different flows or slumps, different finenesses of fine aggregate, etc., are readily discernible by the average person working in concrete. I feel that the indices obtained are consistent, and parallel such recognized differences. For example, see Fig. 6. The concrete made using sand having a fineness modulus of 3.25 was harsh and lacked that particular property desired for the most satisfactory placing. As the fineness modulus was lowered the mixes became more workable. The improvement was nearly proportional to the change in the fineness modulus when based on the index for the concrete using the coarse sand.

Numerous references might be made to show that the flow table is not recognized as a measure of workability under varying conditions of mix and types and gradations of aggregates. However, since the workability is recognized to improve for a given mix and with given materials as the water is increased within reason, the flow may be an indication of the improvement in workability. This will not hold when other conditions of the mix are changed nor can the results be correlated. Although one may obtain the same flow for two different mixes yet the workabilities would be vastly different. To use the time-worn comparison of a rich mix and a lean mix, it is universally recognized that the rich mix will be more workable than the lean although gaged to the same flow or slump. It is within reason that the index obtained for a 1:3:6 mix should be four times

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[1] Engineer, Celite Products Co., Lompoc, Calif.

Mr. Smith. as great as that obtained for a 1:1½:3 mix when both are gaged to a flow of 100 per cent. It is also within reason that the index should be eight times as high when the mixes are gaged to a flow of 120 since so much water was necessary for this flow with the lean mix that the aggregates compacted in the mold indicating exaggerated segregation.

I would point out that, in Fig. 8, the results shown were all obtained with the basic 1:2:4 by volume mix to which the various admixtures were added. This mix was not considered lean and segregation did not have exaggerated effects on the indices.

It is granted, and was pointed out, that the variations of single tests are greater than would be desired. However, one does not look to the range of the results but to the average for a likely value to indicate a probable condition.

Referring to the variations in Fig. 5 for the three mixes noted by Mr. Dwyer, I might say that the probable errors of the three indices, computed from the variation of single results from the average are ± 2.3 to ± 2.4. This represents a probable error of about 1.5 per cent.

In the fourth paragraph under Uniformity of Results, it was pointed out that a duplicate set of tests was made using a new lot of cement. There was no particular reason to doubt that there would be a difference between the cements since other tests indicated them to be nearly equal in every respect. The variations of the indices obtained from the average, amounts to 2.8 per cent for the celite concrete and less than one per cent for the plain concrete. These and other check results we have made indicating equally as good agreement point with reasonable assurance to a reliable index.

The range considered thus far in the study has been ± 33 per cent of cement from a basic 1:2:4 by volume mix. For these variations of the cement content the mix becomes a 1:1½:3 or a 1:3:6. The first is nearly as rich a mix as is met in most jobs but the other, though used rather extensively for certain classes of construction, is so lean that it appears to have lost all semblance of a plastic material. Although the plain 1:3:6 mix is harsh and non-plastic it can be made plastic by the use of reasonable amounts of proper admixtures, so that it is difficult to say whether or not the mix is entirely without the range of plastic mixes.

COMPRESSION, FLEXURE AND TENSION TESTS OF PLAIN CONCRETE

By H. F. Gonnerman[1] and E. C. Shuman[2]

Synopsis

The strength of concrete in bending and direct tension is an important element in the design of concrete roads. The investigation described in this paper was made for the purpose of determining the effect on the flexural and tensile strength of plain concrete of some of the factors which are known to influence the compressive strength. Parallel compression, flexural and tension tests were made at ages of 3 days to 1 year on specimens molded from the same batches of concrete. The variables studied included: Quantity of mixing water, quantity of cement, size, grading and type of aggregate, age at test, curing condition, and size and length of tension specimen.

Compression tests were made on 6 by 12-in. cylinders and flexural tests on 7 by 10 by 38-in. beams loaded at the third points of a 36-in. span. Tension tests were generally made on 6 by 18-in. cylinders; a few tests were made on cylinders of other lengths and diameters. Special grips were employed to hold the tension specimens during loading.

The tests show that the relations between water-cement ratio and the modulus of rupture and tensile strength of plastic workable concrete mixes are similar to the water-cement ratio - strength relation for compression tests, showing that for given materials and methods of manipulation the water-cement ratio is a governing factor in the strength of concrete whether tested in compression, flexure or tension. Typical age-strength relations were obtained with each of the three types of test. An advantage was shown in all three types of testing in the use of a low water-cement ratio in the production of high early strength concrete.

The principal conclusions from the tests are given at the close of the paper.

Introduction

The design of most plain and reinforced concrete structures involves a knowledge of the compressive strength of concrete since one of the principal stresses to be resisted by the concrete is direct compression. In the design of some types of structures, as, for example, concrete roads, the compressive strength of the concrete is generally considered to be of less importance than the strength in bending, and the design of this type of road is based on the flexural strength or modulus of rupture. The strength of concrete in direct tension also is important in road design as it determines the resistance of the road to cracking and thus has a bearing on the location of expansion joints.

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[1] Manager, Research Laboratory, Portland Cement Association, Chicago, Ill.

[2] Assistant Research Engineer, Research Laboratory, Portland Cement Association, Chicago, Ill.

The factors which affect the compressive strength of plain concrete have been quite thoroughly investigated in recent years and much valuable information on this subject is available to designing engineers in the published literature, but comparatively little is known regarding the effect of these factors on the flexural and tensile strength. The tests described in this paper were made to study the effect on the flexural and tensile strengths of concrete of some of the factors which are known to influence the compressive strength and to determine if there is any relationship between these strengths when the specimens are made from the same batches, and tested under identical conditions as to manipulation, curing, etc.

SCOPE OF TESTS

The tests covered a study of the effect of the following factors on the compressive, flexural and tensile strength of plain concrete and of the relationships between these strengths for different conditions of test:

1. Quantity of mixing water;
2. Quantity of cement;
3. Size, grading and type of aggregate;
4. Age at test;
5. Curing conditions;
6. Area in tension; and
7. Length of specimen in tension.

Compression tests were made on 6 by 12-in. cylinders and flexure tests on 7 by 10 by 38-in. beams loaded at the third points of a 36-in. span. Tension tests were generally made on 6 by 18-in. cylinders; a few tests were made on cylinders of other lengths and diameters as indicated in Tables IV and VIII. Specimens of each type were made from the same batch for test at ages of 3, 7 and 28 days, 3 months, 1, 2 and 5 years. In general five specimens of each type were made for each condition of test on different days. This paper gives the results of tests through the 1-year period on 560 compression specimens, 560 beams and 640 tension specimens.

The tests formed a part of the experimental studies on concrete and concrete materials being carried out at the Research Laboratory of the Portland Cement Association, Chicago.

TABLE I.—TESTS OF AGGREGATES.

Sieve Analyses and Unit Weights of Aggregates

Sieve analyses made with Tyler standard screen scale sieves which conformed to the requirements of the A.S.T.M. Standard Method of Test for Sieve Analyses of Aggregates for Concrete (C 41 - 24).

Unit weights of aggregates determined in a $\frac{1}{2}$ cu. ft. measure, using methods specified in A.S.T.M. Standard Method of Test for Unit Weight of Aggregate for Concrete (C 29 - 27).

Aggregate		Sieve Analysis Amount Retained on Each Sieve, per cent by weight									Fineness Modulus[a]	Weight, lb. per cu. ft.
Kind	Size	No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	$\frac{3}{8}$ in.	$\frac{3}{4}$ in.	$1\frac{1}{2}$ in.		
Elgin Sand	0 to No. 14	93	82	35	0	..	..	..	..	..	2.10	104
	0 to No. 8	95	87	52	26	0	..	..	..	..	2.60	108
	0 to No. 4	96	89	60	38	16	1	0	0	0	3.00	112
Elgin Sand and Gravel	0 to $\frac{3}{8}$ in.	97	93	73	59	44	34	0	0	0	4.00	119
	0 to $\frac{3}{4}$ in.	98	95	82	72	62	54	37	0	0	5.00	126
	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	127
	0 to $1\frac{1}{2}$ in.	97	92	70	53	37	26	19	6	0	4.00	121
	0 to $1\frac{1}{2}$ in.	98	94	80	69	58	51	38	12	0	5.00	127
	0 to $1\frac{1}{2}$ in.	99	97	88	82	74	69	52	17	0	5.75	125
	0 to $1\frac{1}{2}$ in.	99	97	90	85	79	75	56	19	0	6.00	123
	0 to $1\frac{1}{2}$ in.	99	98	93	88	84	82	61	20	0	6.25	121
	0 to $1\frac{1}{2}$ in.	100	99	95	92	89	87	66	22	0	6.50	117
Elgin Sand and Chicago Limestone	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	126
Elgin Sand and Ohio Dolomitic Limestone	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	122
Elgin Sand and Wisconsin Granite	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	121
Elgin Sand and Chicago Blast Furnace Slag	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	127
Elgin Sand and New York Siliceous Gravel	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	127
Elgin Sand and New Jersey Trap Rock	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	124
Elgin Sand and Minnesota Sandstone	0 to $1\frac{1}{2}$ in.	98	96	85	77	69	62	47	16	0	5.50	114

[a] Sum of percentages in sieve analysis, divided by 100.

Specific Gravity and Absorption of Aggregates

Specific gravity tests made in accordance with A.S.T.M. Standard Method of Test for Apparent Specific Gravity of Coarse Aggregates (Serial Designation D 30 - 18).

Absorption of aggregates and specific gravity of fine aggregate made in accordance with methods outlined in *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part I, pp. 301 and 302 (1920).

Kind of Aggregate	Weight, lb. per cu. ft.	Specific Gravity		Absorption, per cent by weight				
		True	Apparent	5 minutes	30 minutes	3 hours	6 hours	24 hours
Elgin Sand	112	2.74	2.66	0.9	...	1.2	1.2	1.2
Elgin Gravel	107	2.69	2.62	1.3	1.5	1.5	1.5	1.7
Chicago Limestone	100	2.71	2.64	0.6	0.9	0.9	0.9	1.1
Ohio Dolomitic Limestone	96	2.69	2.54	2.0	2.0	2.0	2.0	2.1
Wisconsin Granite	97	2.64	2.63	0.1	0.1	0.1	0.1	0.1
Chicago Blast Furnace Slag	97	2.67	2.59	0.8	1.1	1.1	1.1	1.3
New York Siliceous Gravel	106	2.65	2.61	0.5	0.6	0.6	0.6	0.6
New Jersey Trap Rock	100	2.94	2.90	0.1	0.1	0.2	0.2	0.4
Minnesota Sandstone	91	2.50	2.23	4.1	4.1	4.0	4.1	4.1

MATERIALS AND METHODS

Aggregates.—In all cases the fine aggregate was Elgin sand graded from 0 to No. 4 sieve. The coarse aggregate generally consisted of Elgin gravel graded from the No. 4 sieve to 1½ in.; in one group of tests other gradings were used. A number of tests were made with other coarse aggregates consisting of crushed limestone and crushed blast-furnace slag from Chicago, crushed granite from Wisconsin and crushed sandstone from Minnesota. In a few of the tension tests a siliceous gravel from New York, a crushed trap rock from New Jersey, and a crushed dolomitic limestone from Ohio were used as coarse aggregates. The coarse aggregates were screened into three sizes and recombined to a pre-determined sieve analysis.

The aggregates were room-dry when used. Determinations were made of the moisture in the aggregates and of the absorption of the aggregates for use in calculating the net water-cement ratios. Sieve analyses and miscellaneous tests of the aggregates are given in Table I.

Portland Cement.—A mixture of equal parts of four brands purchased in Chicago was used in all of the tests.

Mixing Water was from Lake Michigan and was tempered to 70° F. before use.

Mixing Concrete.—The concrete was mixed in a 3½-cu-ft. tilting mixer in batches of about 2.3 cu. ft. The mixing continued for 1 minute after the water had been added. The consistency of the concrete was determined by means of a 30-in. flow table using fifteen ½-in. drops in 10 seconds and in most of the tests was such as to give a flow of 200 per cent ± 10 per cent (slump 2 to 6 in.). In a few tests drier and wetter consistencies were used. The amount of water required to obtain a given consistency when the first round of test specimens was made was maintained constant in subsequent rounds.

In order to secure a direct comparison of the strengths of the three types of specimens, one specimen of each type was made from each batch of concrete. In general, the making of specimens was repeated on each of five different days.

Molding of Specimens.—Cylinders for tension tests 6 in. in diameter and 12, 18, 24 and 30 in. long, and 2 and 10 in. in diameter and 3 diameters long, were molded in steel forms resting on machined metal base plates. The 6 and 10-in. cylinders were molded in layers of about 4 in., each layer being rodded 30 times with a ⅝-in. round bullet-pointed steel rod. The 2-in. cylinders were filled in three equal layers, each layer being rodded 30 times with a ¼-in. round bullet-pointed steel rod. The tension specimens were troweled smooth on top. The 6 by 12-in. cylinders for compression tests were molded in three layers in accordance with A.S.T.M. standard methods and capped with neat cement paste 3 to 4 hours after molding. All cylinders were covered with steel plates to prevent evaporation of water.

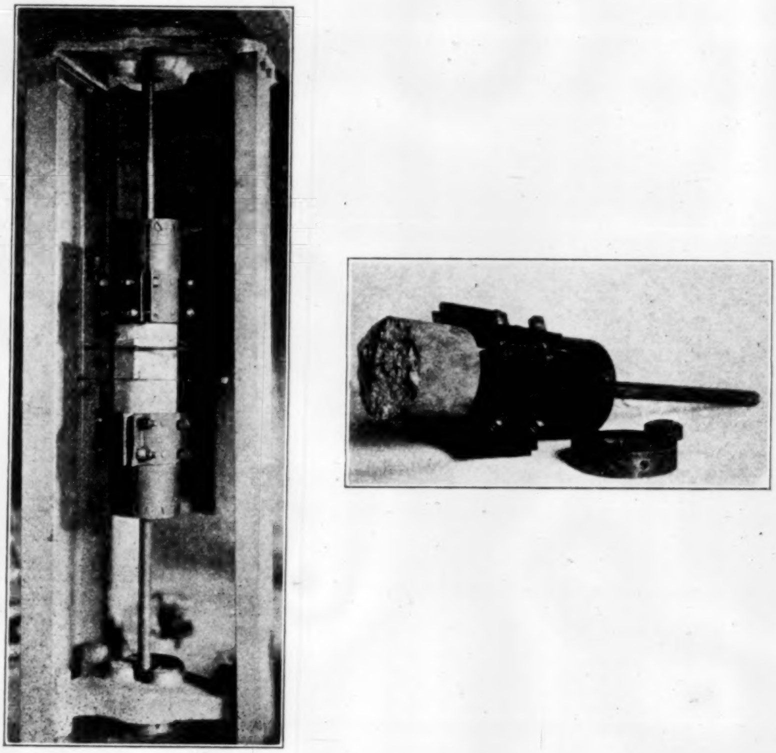

FIG. 1.—Tension Specimen in Testing Machine and Tension Apparatus.

The 7 by 10 by 38-in. beams were molded in two layers in oiled wood forms resting on dampened building paper, each layer being rodded 75 times with a $\frac{5}{8}$-in. round bullet-pointed steel rod. Immediately after molding, the tops of the beams were finished with a wood float and covered with wet burlap to prevent evaporation.

Curing.—Unless otherwise noted, the test specimens were placed in the moist room or in air in the laboratory after 1 day in the molds. Those cured in the moist room were tested damp. In one group of tests the specimens were transferred from the moist room to the air of the laboratory at various ages and tested dry.

Testing.—The tension specimens were tested in a 50,000-lb. motor-driven 3-screw testing machine using special tension grips for loading. Each of these grips (Fig. 1) consisted of a section of steel tubing slit into quadrants and attached to heavy end plates. The specimen was gripped by tightening tangential bolts passing through pieces of channel iron riveted to each of the quadrants. The tubing was lined with leather $\frac{1}{8}$ in. thick in order to secure a uniform grip on the surface of the specimen. Load from the machine was transferred to the grips by bolts passing through the heavy end plates to similar bearing plates on the moving and stationary heads of the testing machine. The nuts holding the bolts were turned to fit spherical seats in each of the four plates. Two inches of the length of the specimens 2 in. in diameter, 5 in. of the specimens 6 in. in diameter, and 10 in. of the specimens 10 in. in diameter were held in each grip.

For the first few tension tests, in order to obtain a uniform bearing on the specimen, the grips were applied loosely, the specimen then mounted in the testing machine and a slight load applied to cause slipping of the grips. The grips were then tightened to prevent further slipping during loading. Although this method was satisfactory, it was found that the tests could be made more quickly and with equally good results if the two grips were attached to the specimens simultaneously and each set of bolts tightened gradually.

The beams were tested as molded in the 50,000-lb. machine used for the tension tests and were loaded at the third points of a 36-in. span.

Compression tests were made in a motor-driven 300,000-lb. 2-screw testing machine using a spherical block on top of the specimens.

The head of the testing machine moved under load at a rate of about 0.025 in. per minute in all of the tests.

DATA AND DISCUSSION OF TESTS

Data of Tests.—The principal data of the strength tests are given in Tables II and VIII. The headings of the tables give detailed information on the specimens, materials and methods used. Some of the important relationships are shown in Figs. 2 to 8.

Water-Cement Ratio-Strength Relations.—One of the most important factors affecting the strength and other properties of concrete is the amount of mixing water which is generally expressed as a ratio by volume to the volume of the cement, or the "water-cement ratio." There is considerable published information on the effect of quantity of mixing water on the compressive strength of

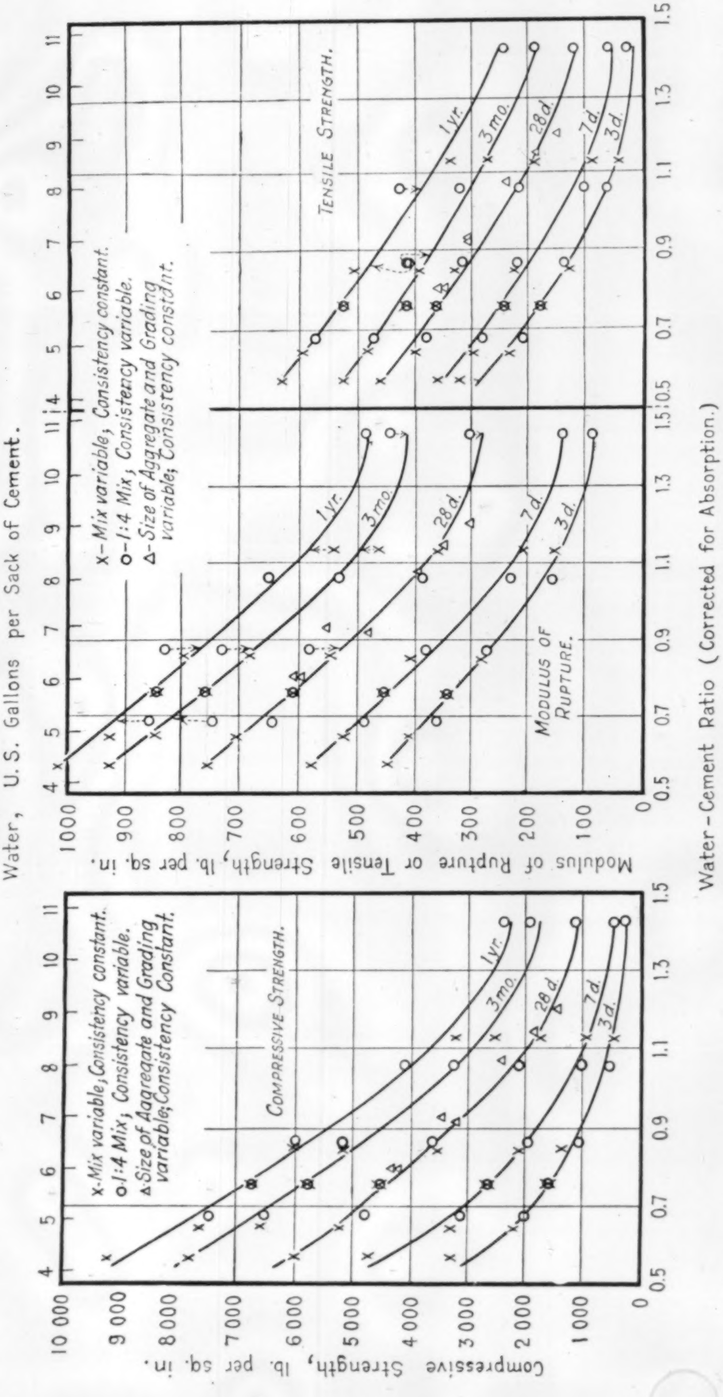

FIG. 2.—Relations Between Compressive Strength, Modulus of Rupture, and Tensile Strength of Concrete and Water Content.

Data from Tables II to IV.
Compression tests of 6 by 12-in. cylinders.

Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.

Aggregate: sand and gravel from Elgin, Ill.
Specimens cured in moist room; tested damp.

concrete but few data are available showing its effect on flexural and tensile strength.

Curves showing water-cement ratio-strength relations at ages of 3 days to 1 year for the three types of test used in this investigation are given in Fig. 2. These curves for the most part are based on data in Tables II and III; data from Table IV are included on the diagrams for the 28-day tests. The curves show the relation between water-

TABLE II.—STRENGTHS OF CONCRETE FOR DIFFERENT WATER CONTENTS.

Compression tests of 6 by 12-in. concrete cylinders.
Flexural tests of 7 by 10 by 38-in. concrete beams loaded at the third points of a 36-in. span.
Tension tests of 6 by 18-in. concrete cylinders.
Mix 1:4 by volume.
Aggregate: Elgin sand and gravel graded 0 to 1½ in. (Fineness Modulus, 5.50).
Cement: A mixture of equal parts of four brands of portland cement purchased in Chicago.
Consistency determined by means of a 30-in. flow table using fifteen ½-in. drops in 10 seconds; flow 200 per cent ± 10. per cent for relative water content of about 1.10; concrete of relative water content 0.80 and 0.90 non-workable.
Concrete mixed for 1 minute in a 3½ cu. ft. tilting mixer in batches of sufficient size to make one of each of the three types of specimen.
Concrete placed in layers of about 4 in., each layer rodded 30 times with a bullet-pointed steel rod; sides of beam spaded with trowel.
Specimens removed from forms after 1 day and cured in moist room until test: tested damp.
Each value is the average of 5 tests made on different days unless otherwise noted.

Relative Water Content	Water-Cement Ratio		Average Flow, per cent	Weight of Concrete lb. per cu. ft.	Cement, sacks per cu. yd.	Strengths, lb. per sq. in					Ratios									
	Nominal	Net				3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year
						COMPRESSIVE STRENGTH					COMPRESSIVE STRENGTH TO TENSILE STRENGTH					COMPRESSIVE STRENGTH TO MODULUS OF RUPTURE				
0.80	0.60	0.53	...	143	6.1	1510	1880	2430	3120	3640	15.1	13.5	14.3	15.6	13.7	10.8	8.5	9.7	8.7	8.9
0.90	0.67	0.61	119	151	6.4	2140	3300	4380	6140	7010	12.6	13.0	14.6	15.9	13.9	8.3	9.0	9.4	10.5	11.0
1.00	0.74	0.68	161	152	6.4	2000	3100	4710	6460	7420	9.5	11.3	12.4	13.7	13.0	5.7	6.5	7.4	8.8	8.7
1.10	0.82	0.76	208	152	6.3	1560	2620	4460[a]	5710	6720	8.9	10.9	12.4	13.7	12.8	4.7	5.9	7.4	7.6	8.0
1.25	0.93	0.87	257	160	6.2	1070	1900	3560	5130	6000	7.9	8.6	11.2	12.3	14.0	4.0	5.1	6.1	7.1	7.3
1.50	1.12	1.06	[b]	149	6.0	535	1020	2090	3160	4050	8.9	10.2	9.7	9.9	9.5	3.6	4.5	5.5	6.0	6.3
2.00	1.49	1.43	[b]	148	5.7	255	480	1120	1900	2350	8.5	...	9.3	10.3	9.8	3.0	3.6	3.7	4.3	4.8
						MODULUS OF RUPTURE					MODULUS OF RUPTURE TO TENSILE STRENGTH					MODULUS OF RUPTURE TO COMPRESSIVE STRENGTH				
0.80	0.60	0.53	...	143	6.1	140	220	250	360	410	1.40	1.57	1.47	1.80	1.54	0.093	0.118	0.103	0.115	0.112
0.90	0.67	0.61	119	151	6.4	260	365	465	585	635	1.54	1.43	1.55	1.52	1.25	0.121	0.111	0.106	0.095	0.091
1.00	0.74	0.68	161	152	6.4	355	480	640	735	850	1.70	1.75	1.69	1.56	1.48	0.177	0.154	0.135	0.114	0.114
1.10	0.82	0.76	208	152	6.3	335	445	605[a]	755	840	1.92	1.85	1.69	1.82	1.61	0.214	0.170	0.135	0.132	0.125
1.25	0.93	0.87	257	150	6.2	265	375	580	725	825	1.96	1.70	1.85	1.75	1.92	0.248	0.196	0.164	0.141	0.132
1.50	1.12	1.06	[b]	149	6.0	150	225	380	530	645	2.50	2.25	1.77	1.65	1.52	0.280	0.222	0.182	0.167	0.159
2.00	1.49	1.43	[b]	148	5.7	85	135	300	440	485	2.84	...	2.50	2.38	2.02	0.334	0.281	0.270	0.232	0.208
						TENSILE STRENGTH					TENSILE STRENGTH TO COMPRESSIVE STRENGTH					TENSILE STRENGTH TO MODULUS OF RUPTURE				
0.80	0.60	0.53	...	143	6.1	100	140	170	200	265	0.066	0.074	0.070	0.064	0.073	0.71	0.64	0.68	0.56	0.65
0.90	0.67	0.61	119	151	6.4	170	255	300	385	505	0.079	0.077	0.068	0.063	0.072	0.65	0.70	0.64	0.66	0.80
1.00	0.74	0.68	161	152	6.4	210	275	380	470	575	0.105	0.089	0.081	0.073	0.077	0.59	0.57	0.59	0.64	0.72
1.10	0.82	0.76	208	152	6.3	175	240	360[a]	415	525	0.112	0.092	0.081	0.073	0.078	0.52	0.54	0.59	0.55	0.62
1.25	0.93	0.87	257	150	6.2	135	220	315	415	430	0.126	0.116	0.089	0.081	0.072	0.51	0.59	0.54	0.57	0.52
1.50	1.12	1.06	[b]	149	6.0	60	100	215	320	425	0.112	0.098	0.103	0.101	0.105	0.40	0.44	0.57	0.60	0.66
2.00	1.49	1.43	[b]	148	5.7	30	...	120	185	240	0.118	...	0.107	0.097	0.102	0.35	...	0.40	0.42	0.49

[a] Average of 15 tests.
[b] Flows beyond limit of flow table.

cement ratio and strength of moist-cured concrete for (1) different mixes of the same consistency, (2) a 1:4 mix of different consistencies, and (3) a 1:4 mix of a given consistency in which the size and grading of aggregate varied. In plotting the diagrams, the strengths of concretes which were unworkable due to insufficient mixing water or to under-sanding were omitted.

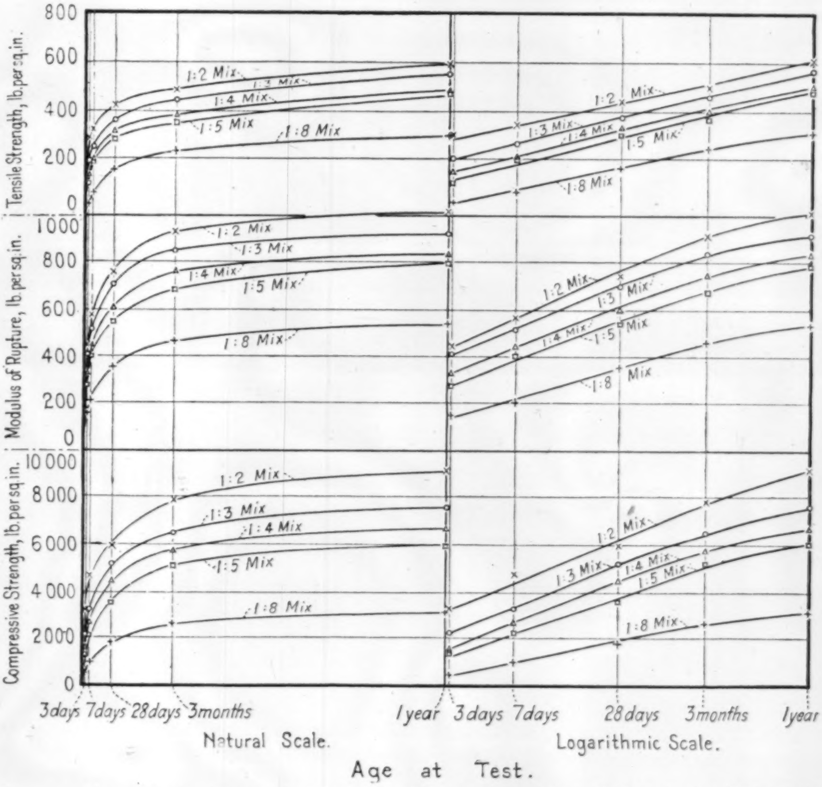

FIG. 3.—Compressive Strength, Modulus of Rupture and Tensile Strength of Concrete for Different Mixes.

Data from Table III.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Aggregate: sand and gravel from Elgin, Ill.; graded 0 to $1\frac{1}{2}$ in. (Fineness Modulus, 5.50).
Consistency constant; flow 200 per cent ± 10 per cent.
Specimens cured in moist room; tested damp.

It will be noted that the water-cement ratio - strength curves for the flexure and tension tests are similar to those obtained in the compression tests. The results are quite consistent at all ages, the plotted points in most cases falling close to the curves showing that for plastic, workable mixes and moist curing the water-cement ratio is a governing factor in the strength of concrete whether it be tested in compression, flexure or tension.

TABLE III.—STRENGTHS OF CONCRETE FOR DIFFERENT MIXES.

Compression tests of 6 by 12-in. concrete cylinders.
Flexural tests of 7 by 10 by 38-in. concrete beams loaded at the third points of a 36-in. span.
Tension tests of 6 by 18-in. concrete cylinders.
Aggregate: Elgin sand and gravel graded 0 to 1½ in. (Fineness Modulus, 5.50).
Flow of concrete 200 per cent ± 10; relative water content about 1.10.
Each value is the average of 5 tests unless otherwise indicated.
For further details of tests see notes accompanying Table II.

Mix by Volume	Water-Cement Ratio		Weight of Concrete lb. per cu. ft.	Cement, sacks per cu yd.	Strengths, lb. per sq. in.					Ratios									
	Nominal	Net			3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year
					COMPRESSIVE STRENGTH					COMPRESSIVE STRENGTH TO TENSILE STRENGTH					COMPRESSIVE STRENGTH TO MODULUS OF RUPTURE				
1:2	0.60	0.57	150	10.5	3220	4650	5960	7810	9170	9.9	12.8	12.8	15.0	14.5	7.25	8.10	7.95	8.45	9.0
1:3	0.69	0.64	150	7.8	2160	3250	5140	6500	7570	9.4	11.1	12.8	13.5	12.7	5.27	6.33	7.30	7.70	8.2
1:4	0.82	0.76	152	6.3	1560	2620	4460[a]	5710	6720	8.9	10.9	12.5	13.9	12.8	4.67	5.90	7.40	7.60	8.0
1:5	0.93	0.85	152	5.2	1340	2120	3510	5130	6010	10.3	9.4	10.8	13.2	11.8	4.87	5.23	6.45	7.60	7.6
1:8	1.26	1.13	151	3.4	480	945	1740	2550	3140	12.0	11.1	9.4	9.4	9.4	3.20	4.61	4.90	5.55	5.8
					MODULUS OF RUPTURE					MODULUS OF RUPTURE TO COMPRESSIVE STRENGTH					MODULUS OF RUPTURE TO TENSILE STRENGTH				
1:2	0.60	0.57	150	10.5	445	575	750	925	1015	0.138	0.123	0.126	0.118	0.111	1.37	1.59	1.64	1.76	1.61
1:3	0.69	0.64	150	7.8	410	515	705	845	925	0.190	0.158	0.137	0.130	0.122	1.78	1.78	1.76	1.76	1.56
1:4	0.82	0.76	152	6.3	335	445	605[a]	755	840	0.214	0.170	0.135	0.132	0.125	1.92	1.85	1.67	1.82	1.61
1:5	0.93	0.85	152	5.2	275	405	545	680	795	0.205	0.191	0.155	0.132	0.132	2.13	1.80	1.67	1.75	1.56
1:8	1.26	1.13	151	3.4	150	205	355	460	540	0.312	0.217	0.204	0.180	0.172	3.75	2.41	1.92	1.70	1.61
					TENSILE STRENGTH					TENSILE STRENGTH TO COMPRESSIVE STRENGTH					TENSILE STRENGTH TO MODULUS OF RUPTURE				
1:2	0.60	0.57	150	10.5	325	360	460	525	630	0.101	0.078	0.078	0.067	0.069	0.73	0.63	0.61	0.57	0.62
1:3	0.69	0.64	150	7.8	230	290	400	480	595	0.106	0.090	0.078	0.074	0.079	0.56	0.56	0.57	0.57	0.64
1:4	0.82	0.76	152	6.3	175	240	360[a]	415	525	0.112	0.092	0.080	0.072	0.078	0.52	0.54	0.60	0.55	0.62
1:5	0.93	0.85	152	5.2	130	225	325	390	510	0.097	0.106	0.093	0.076	0.085	0.47	0.55	0.60	0.57	0.64
1:8	1.26	1.13	151	3.4	40	85	185	270	335	0.083	0.090	0.106	0.106	0.107	0.27	0.41	0.52	0.59	0.62

[a] Average of 15 tests.

The following typical values of compressive and tensile strength and of modulus of rupture at 7 and 28 days for four water-cement ratios, taken from the curves in Fig. 2, show the extent to which an increase in the quantity of mixing water reduced the strengths:

Water-Cement Ratio	Water, gal. per sack	Compressive Strength, lb. per sq. in.		Modulus of Rupture, lb. per sq. in.		Tensile Strength, lb. per sq. in.	
		7 days	28 days	7 days	28 days	7 days	28 days
0.60	4.5	4000	5700	550	725	330	440
0.80	6.0	2300	4000	410	580	230	340
1.00	7.5	1300	2600	280	430	140	240
1.20	9.0	800	1600	180	325	80	170

Age-Strength Relations for Moist-Cured Concrete.—Figure 3, based on data in Table III, shows the relations between age at test and the compressive, flexural and tensile strength of moist-cured concrete for five mixes ranging from 1:2 to 1:8 by volume. The concretes were made from Elgin sand and gravel and were mixed to the same consistency. The cement content ranged from 10.5 sacks per cu. yd. for the 1:2 mix to 3.4 sacks per cu. yd. for the 1:8 mix. Figure 4, based on data in Table II, shows similar curves for a 1:4 mix of variable consistency ranging from a fairly stiff mix of relative water content 1.00 (slump $1\frac{1}{4}$ in.) to a very wet mix of relative water con-

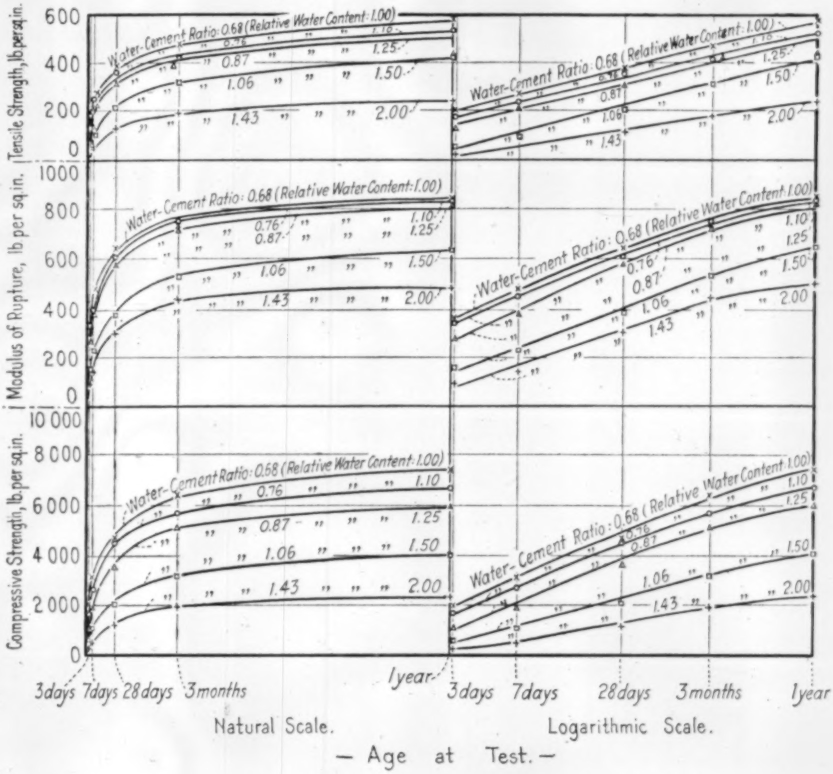

FIG. 4.—Compressive Strength, Modulus of Rupture and Tensile Strength of Concrete for Different Water Contents.

Data from Table II.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Aggregate: sand and gravel from Elgin, Ill.; graded 0 to $1\frac{1}{2}$ in. (Fineness Modulus, 5.50).
Mix 1:4 by volume.
Specimens cured in moist room until test; tested damp.

tent 2.00. The curves for dry, non-plastic mixes of relative water contents 0.80 and 0.90 were omitted from this figure to avoid confusion. For purpose of comparison and in order to show the strengths at the early ages more clearly, the age-strength diagrams have been plotted with age at test drawn both to natural and logarithmic scales. The diagrams in Figs. 3 and 4 for all three types of test are similar and show typical age-strength relations for moist-cured concrete. In all cases the strengths increased with age.

TABLE IV.—STRENGTHS OF CONCRETE FOR DIFFERENT SIZES AND GRADINGS OF AGGREGATE.

Compression tests of 6 by 12-in. concrete cylinders.
Flexural tests of 7 by 10 by 38-in. concrete beams loaded at the third points of a 36-in. span.
Tension tests of 2, 6 and 10-in. diameter concrete cylinders, 3 diameters long.
Age at test, 28 days.
Mix 1:4 by volume.
Aggregate: Elgin sand and pebbles of the gradings indicated; for sieve analyses see Table I.
One specimen of each size and type made from a single batch.
Flow 200 per cent ± 10 per cent; relative water content about 1.10.
Each value is the average of 5 tests made on different days unless otherwise noted.
For further details of tests see notes accompanying Table II.

Aggregate			Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Cement, sacks per cu. yd.	Strengths, lb. per sq. in.					Ratio, Tensile Strength of 6 by 18-in. Cylinders to Compressive Strength	Ratio, Tensile Strength of 6 by 18-in. Cylinders to Modulus of Rupture	Ratio, Modulus of Rupture to Compressive Strength
Size	Fineness Modulus	Weight, lb. per cu. ft.	Nominal	Net			Compressive	Modulus of Rupture	Tensile					
									2 by 6-in. Cylinder	6 by 18-in. Cylinder	10 by 30-in. Cylinder			
0 to No. 14	2.10	104	1.23	1.20	133	6.2	1450	295	130	145	150	0.100	0.49	0.204
0 to No. 8	2.60	108	1.17	1.14	136	6.1	1850	340	190	185	180	0.100	0.54	0.184
0 to No. 4	3.00	112	1.10	1.07	141	6.2	2380	395	265	230	205	0.097	0.58	0.166
0 to $\frac{3}{8}$ in.	4.00	119	0.97	0.93	146	6.3	3420	550	315	310	290	0.091	0.56	0.161
0 to $\frac{3}{4}$ in.	5.00	126	0.86	0.80	151	6.3	4260	590	...	350	340	0.082	0.59	0.138
0 to $1\frac{1}{2}$ in.	5.50	127	0.82	0.76	152	6.3	4460	605	...	360	355	0.081	0.59	0.135
0 to $1\frac{1}{2}$ in.	4.00	121	0.97	0.92	145	6.1	3240	475	...	310	...	0.096	0.65	0.146
0 to $1\frac{1}{2}$ in.	5.00	127	0.86	0.80	149	6.1	4220	600	...	355	...	0.084	0.59	0.142
0 to $1\frac{1}{2}$ in.	5.75	125	0.80	0.74	152	6.4	4340	610	...	330	...	0.076	0.54	0.140
0 to $1\frac{1}{2}$ in.	6.00	123	0.78	0.71	153	6.5	4340	670	...	305	...	0.070	0.46	0.154
0 to $1\frac{1}{2}$ in.	6.25	121	0.75	0.68	153	6.6	4100	680	...	280	...	0.068	0.41	0.166
0 to $1\frac{1}{2}$ in.	6.50	117	0.74	0.67	152	6.8	3610	630	...	240	...	0.067	0.38	0.174

In Table V, the strength at any period in terms of the strengths at other periods up to 1 year are shown. This tabulation is useful in estimating the probable strength of concrete at any age from a test at some other age when the conditions of treatment are comparable to those used in this investigation. The percentages in this table are based on strengths taken from the curves in Fig. 2 for six water-cement ratios. The data show that the lower the water-cement ratio, the higher are the strengths relatively at the early ages. The data also show that the flexural strength is relatively higher at the early ages

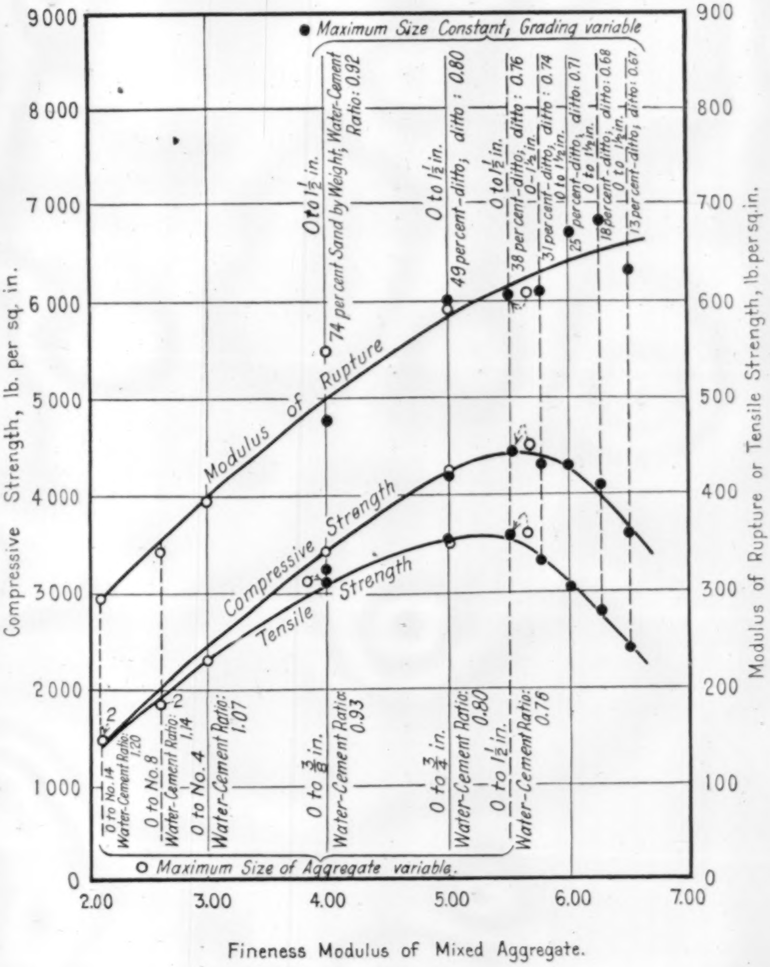

FIG. 5.—Compressive Strength, Modulus of Rupture and Tensile Strength of Concrete for Different Sizes and Gradings of Aggregate.

Data from Table IV.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Mix 1:4 by volume.
Aggregate: sand and gravel from Elgin, Ill.
Consistency constant; flow 200 per cent ± 10 per cent.
Age at test, 28 days.
Specimens cured in moist room; tested damp.

than the compressive; the tensile strength is also relatively higher than the compressive at the early ages, except for the 1.07 and 1.20 water-cement ratios. A concrete mix containing about 6 gal. of water per sack of cement is frequently employed in road construction. For such a mix typical percentages of the 28-day strengths for the three types of test are as follows:

TYPE OF TEST	AGE				
	3 DAYS	7 DAYS	28 DAYS	3 MONTHS	1 YEAR
Compression	35	59	100	135	161
Flexure	53	71	100	126	143
Tension	46	68	100	121	150

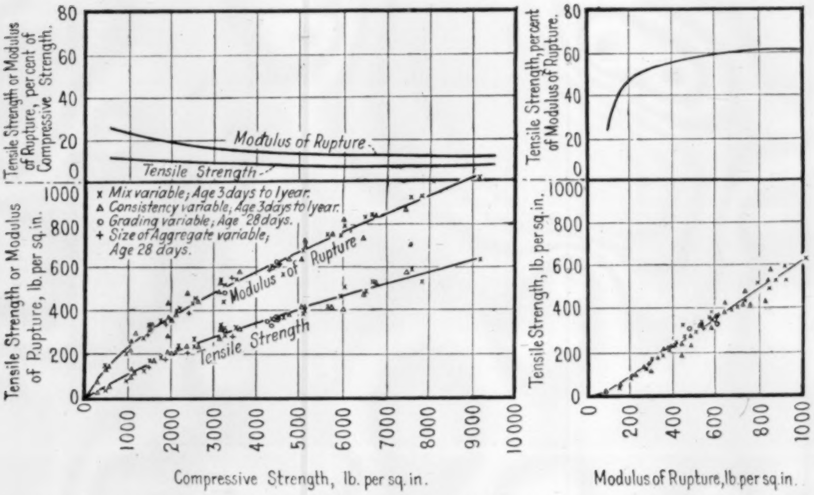

FIG. 6.—Relations Between Compressive Strength, Modulus of Rupture and Tensile Strength of Concrete.

Data from Tables II to IV.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Aggregate: sand and gravel from Elgin, Ill.
Specimens cured in moist room; tested damp.

Age-strength curves for concrete cured under different conditions are given in Fig. 8 and are discussed later.

Effect of Size and Grading of Aggregate.—Data from Table IV for two groups of tests of 1:4 concrete are plotted in Fig. 5. In one group of tests the maximum size of aggregate was varied and ranged from particles passing the No. 14 sieve to particles passing the 1½-in. sieve (fineness modulus 2.10 to 5.50). In the other group, the maximum size was 1½ in. and the grading of the mixed aggregate was varied by changing the amount of sand in the mixture which ranged from

74 to 13 per cent by weight (fineness modulus 4.00 to 6.50). This 1:4 mix became harsh-working when the percentage of sand in the mixed aggregate was less than about 38 per cent (fineness modulus about 5.50).

TABLE V.—RELATIVE STRENGTHS OF MOIST-CURED CONCRETE AT DIFFERENT AGES.

Values are based on strengths taken from curves in Fig. 2 and are ratios of the strengths at each age to the strengths at a given age expressed as percentages.

Strength at Age Indicated Taken as 100 per cent	Compressive Strength, 6 by 12-in. Cylinders					Modulus of Rupture, 7 by 10 by 38-in Beams					Tensile Strength, 6 by 18-in. Cylinders				
	3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year
NET WATER-CEMENT RATIO, 0.60; 4.5 GAL. PER SACK															
3 days	100	160	228	296	340	100	131	170	210	233	100	122	163	189	228
7 days	63	100	143	185	212	77	100	131	161	178	82	100	133	155	186
28 days	44	70	100	130	149	59	77	100	123	136	61	75	100	116	140
3 months	34	54	77	100	115	48	62	81	100	110	53	65	86	100	121
1 year	29	47	67	87	100	43	56	73	90	100	44	54	72	83	100
NET WATER-CEMENT RATIO, 0.67; 5 GAL. PER SACK															
3 days	100	165	255	335	385	100	130	174	217	240	100	131	178	210	255
7 days	61	100	155	203	233	77	100	133	167	185	76	100	135	161	195
28 days	39	65	100	131	151	58	75	100	125	138	56	74	100	119	144
3 months	30	49	76	100	115	46	60	80	100	110	47	62	84	100	121
1 year	26	43	66	87	100	42	54	72	90	100	39	51	70	83	100
NET WATER-CEMENT RATIO, 0.80; 6 GAL. PER SACK															
3 days	100	168	286	386	460	100	134	188	238	269	100	148	219	264	329
7 days	60	100	170	230	274	74	100	140	177	200	67	100	148	178	222
28 days	35	59	100	135	161	53	71	100	126	143	46	68	100	121	150
3 months	26	44	74	100	120	42	57	79	100	113	38	56	83	100	124
1 year	22	36	62	84	100	37	50	70	88	100	30	45	67	80	100
NET WATER-CEMENT RATIO, 0.93; 7 GAL. PER SACK															
3 days	100	179	326	447	548	100	138	206	266	306	100	170	270	350	450
7 days	56	100	182	250	306	72	100	149	192	221	59	100	159	206	264
28 days	31	55	100	137	168	49	67	100	129	148	37	63	100	130	167
3 months	22	40	73	100	122	38	52	78	100	115	29	49	77	100	129
1 year	18	33	60	82	100	33	45	67	87	100	22	38	60	78	100
NET WATER-CEMENT RATIO, 1.07; 8 GAL. PER SACK															
3 days	100	183	358	517	650	100	147	244	328	384	100	192	358	491	633
7 days	55	100	195	282	354	68	100	166	223	262	52	100	187	256	330
28 days	28	51	100	144	181	41	60	100	135	158	28	54	100	137	177
3 months	19	36	69	100	126	30	45	74	100	117	20	39	73	100	129
1 year	15	28	55	80	100	26	38	63	85	100	16	30	57	78	100
NET WATER-CEMENT RATIO, 1.20; 9 GAL. PER SACK															
3 days	100	188	375	600	750	100	150	275	391	458	100	200	425	625	825
7 days	53	100	200	320	400	67	100	183	256	300	50	100	212	312	412
28 days	27	50	100	160	200	36	55	100	140	164	23	47	100	147	194
3 months	16	31	63	100	125	26	38	70	100	117	16	32	68	100	132
1 year	13	25	50	80	100	22	33	60	85	100	12	24	52	76	100

As may be seen from Table IV, the amount of mixing water required for a given consistency in these two groups of tests was greater for the aggregate mixtures having the smaller maximum size and the larger proportion of fine to coarse, and the strengths were reduced accordingly.

An interesting point is brought out by the curves in Fig. 5. It will be noted that both the tensile and compressive strengths decrease for a fineness modulus greater than 5.50, whereas, the flexural strength continues to increase. In view of these results, flexural tests for field control of concrete quality, particularly with coarsely graded aggregates, should be used with some caution since tests of undersanded or honey-combed and porous concrete might, through the higher flexural strengths, indicate a better quality of concrete than was actually being obtained.

Strength Relations for the Three Types of Test.—Tables II, III, IV, VI and VII give the ratios of the strengths obtained with each of the three types of test to the strengths obtained with the other two types under similar conditions of test. The relations between the strengths may best be seen from the curves in Fig. 6 which are based on data in Tables II to IV except that tests of the two driest consistencies in Table II (relative water content 0.80 and 0.90) and of the undersanded mixtures in Table IV (fineness modulus greater than 5.50), which were unworkable, have been omitted.

The curves in Fig. 6 show well-defined relationships between compressive strength and both modulus of rupture and tensile strength as well as between modulus of rupture and tensile strength. It will be noted that the relations between the compressive strength and modulus of rupture, and between the compressive and tensile strengths are not constant but are nearly so for compressive strengths greater than about 4000 lb. per sq. in. The relation between modulus of rupture and tensile strength is practically constant.

The following tabulation gives a direct comparison between the various strength values from Fig. 6 which covers a wide range in conditions of test:

Strength of Plain Concrete, lb. per sq. in.			Ratio, Modulus of Rupture to Compressive Strength, per cent	Ratio, Tensile Strength to Compressive Strength, per cent	Ratio, Tensile Strength to Modulus of Rupture, per cent
Compressive	Modulus of Rupture	Tensile			
1000	230	110	23.0	11.0	48
2000	375	200	18.8	10.0	53
3000	485	275	16.2	9.2	57
4000	580	340	14.5	8.5	59
5000	675	400	13.5	8.0	59
6000	765	460	12.8	7.7	60
7000	855	520	12.2	7.4	61
8000	930	580	11.6	7.2	62
9000	1010	630	11.2	7.0	63

For concretes having compressive strengths of from 2000 to 5000 lb. per sq. in., the modulus of rupture ranged from about one-fifth to one-seventh of the compressive strength, the tensile strength from about one-tenth to one-twelfth of the compressive strength and from about 0.5 to 0.6 of the modulus of rupture.

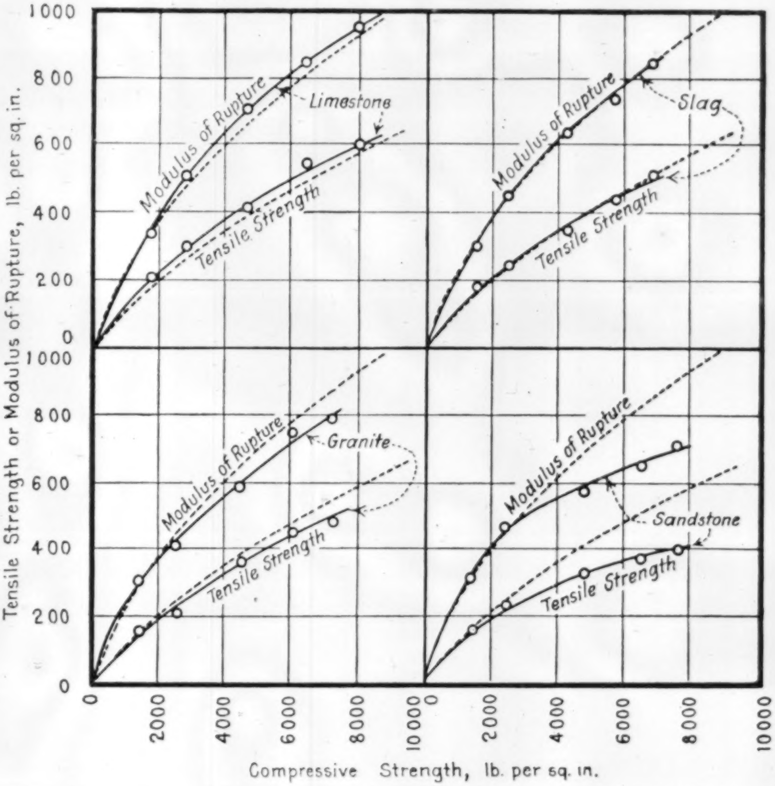

FIG. 7.—Relations of Modulus of Rupture and Tensile Strength to Compressive Strength of Concrete Made from Different Coarse Aggregates.

Data from Table VI. Dash-line curves are for Elgin sand and gravel and are the same as those in Fig. 6.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Fine aggregate: Elgin sand; mixed aggregate graded 0 to $1\frac{1}{2}$ in. (Fineness Modulus, 5.50).
Mix 1:4 by volume.
Consistency constant; flow 220 per cent ± 10 per cent.
Age at test, 3 days to 1 year.
Specimens cured in moist room; tested damp.

Tests with Different Coarse Aggregates.—Table VI and Fig. 7 give data of tests on concretes made from different coarse aggregates. In these tests the mix, fine aggregate, grading and consistency were maintained constant. Miscellaneous tests on the coarse aggregates used are given in Table I.

Concretes from Elgin gravel and Chicago blast-furnace slag had about the same compressive, flexural and tensile strengths at all ages. The Chicago limestone concrete averaged about 11 per cent stronger in compression and flexure and about 20 per cent stronger in tension than the Elgin gravel or Chicago slag concrete for identical conditions of test. In the curves of Fig. 7, these differences appear somewhat less because the dash-line curves of Fig. 7, representing the gravel concrete, cover a wider range of conditions.

TABLE VI.—STRENGTHS OF CONCRETE FROM DIFFERENT COARSE AGGREGATES.

Compression tests of 6 by 12-in. concrete cylinders.
Flexural tests of 7 by 10 by 38-in. concrete beams loaded at third points of a 36-in. span.
Tension tests of 6 by 18-in. concrete cylinders.
Mix 1:4 by volume.
Aggregates graded 0 to 1½ in. (Fineness Modulus, 5.50). Fine aggregate Elgin sand.
Flow 200 per cent ± 10 per cent; relative water content about 1.10.
In general each value is the average of 5 tests made on different days, unless otherwise noted.
For further details of tests see notes accompanying Table II.

Kind of Coarse Aggregate	Water-Cement Ratio		Cement, sacks per cu. yd.	Weight, lb. per cu. ft.		Strengths, lb. per sq. in.					Ratios				
	Nominal	Net		Mixed Aggregate	Concrete	3 days	7 days	28 days	3 mos.	1 year	3 days	7 days	28 days	3 mos.	1 year
						COMPRESSIVE STRENGTH					TENSILE STRENGTH TO COMPRESSIVE STRENGTH				
Elgin Gravel	0.82	0.76	6.3	127	152	1560	2620	4460[a]	5710	6720	0.112	0.092	0.081	0.073	0.078
Chicago Limestone	0.82	0.79	6.3	126	152	1670	2910	4750	6500	8030	0.123	0.101	0.087	0.083	0.073
Chicago Blast Furnace Slag	0.82	0.78	6.2	127	152	1560	2420	4310	5800	6850	0.112	0.097	0.078	0.075	0.075
Wisconsin Granite	0.82	0.80	6.4	121	149	1340	2460	4480	6020	7240	0.104	0.087	0.079	0.072	0.065
Minnesota Sandstone	0.96	0.80	6.3	114	142	1340	2420	4870	6560	7650	0.101	0.093	0.066	0.055	0.050
						MODULUS OF RUPTURE					MODULUS OF RUPTURE TO COMPRESSIVE STRENGTH				
Elgin Gravel	0.82	0.76	6.3	127	152	335	445	605[a]	755	840	0.215	0.170	0.135	0.132	0.125
Chicago Limestone	0.82	0.79	6.3	126	152	335	510	705	835	945	0.200	0.175	0.149	0.128	0.118
Chicago Blast Furnace Slag	0.82	0.78	6.2	127	152	310	450	625	730	835	0.198	0.185	0.145	0.127	0.122
Wisconsin Granite	0.82	0.80	6.4	121	149	295	410	580	740	775	0.220	0.167	0.130	0.123	0.107
Minnesota Sandstone	0.96	0.80	6.3	114	142	305	455	565	645	690	0.228	0.189	0.116	0.098	0.090
						TENSILE STRENGTH					TENSILE STRENGTH TO MODULUS OF RUPTURE				
Elgin Gravel	0.82	0.76	6.3	127	152	175	240	360[a]	415	525	0.52	0.54	0.60	0.55	0.62
Chicago Limestone	0.82	0.79	6.3	126	152	205	295	415	540	590	0.61	0.58	0.59	0.65	0.62
Chicago Blast Furnace Slag	0.82	0.78	6.2	127	152	175	235	335	435	515	0.56	0.52	0.54	0.60	0.62
Wisconsin Granite	0.82	0.80	6.4	121	149	140	215	355	435	470	0.47	0.52	0.61	0.59	0.61
Minnesota Sandstone	0.96	0.80	6.3	114	142	135	225	320	360	385	0.44	0.50	0.57	0.56	0.56
New York Siliceous Gravel	0.78	0.75	6.3	127	152	175	235	320	380	425	...	...	...	...	...
Ohio Dolomitic Limestone	0.78	0.70	6.5	122	151	160	220	320	460	515	...	...	...	...	...
New Jersey Trap Rock	0.78	0.76	6.6	124	157	140	230	350	470	490	...	...	...	...	...

[a] Average of 15 tests.

Concrete from Wisconsin granite or Minnesota sandstone gave somewhat lower compressive strengths at the early ages than the other coarse aggregates but about the same compressive strengths at the later ages. The moduli of rupture and tensile strengths of concrete from the sandstone and granite were generally lower at all ages than

TABLE VII.—STRENGTHS OF CONCRETE FOR DIFFERENT CONDITIONS OF CURING.

Compression tests of 6 by 12-in. concrete cylinders.
Flexural tests of 7 by 10 by 38-in. concrete beams loaded at third points of a 36-in. span.
Tension tests of 6 by 18-in. concrete cylinders.
Mix 1:4 by volume.
Aggregates graded 0 to $1\frac{1}{2}$ in. (Fineness Modulus, 5.50).
Flow 200 per cent ± 10 per cent; relative water content about 1.10.
Water-cement ratio:
  Gravel concrete; nominal 0.82, net 0.76.
  Limestone concrete; nominal 0.82, net 0.78.
Each value is the average of 5 tests made on different days.
For further details of tests see notes accompanying Table II.

Kind of Aggregate	Curing	Strengths, lb. per sq. in.					Ratios				
		3 days	7 days	28 days	3 mos	1 year	3 days	7 days	28 days	3 mos.	1 year
		COMPRESSIVE STRENGTH					TENSILE STRENGTH TO COMPRESSIVE STRENGTH				
Elgin Sand and Gravel	Moist room until test	1560	2620	4460	5710	6720	0.112	0.092	0.081	0.073	0.078
	1 day moist; remainder air of laboratory	1530	2540	4070	4380	4090	0.101	0.093	0.078	0.094	0.090
	7 days moist; remainder air of laboratory	....	....	4790	5450	4890	....	....	0.079	0.086	0.096
	28 days moist; remainder air of laboratory	....	....	....	6080	5930	....	....	....	0.081	0.083
Elgin Sand and Chicago Limestone	Moist room until test	1670	2910	4750	6500	8030	0.123	0.101	0.087	0.083	0.073
	1 day moist; remainder air of laboratory	1740	2830	4140	4450	4230	0.109	0.097	0.091	0.100	0.090
	7 days moist; remainder air of laboratory	....	....	4880	5490	5390	....	....	0.076	0.096	0.097
	28 days moist; remainder air of laboratory	....	....	....	6270	6210	....	....	....	0.088	0.092
		MODULUS OF RUPTURE					MODULUS OF RUPTURE TO COMPRESSIVE STRENGTH				
Elgin Sand and Gravel	Moist room until test	335	445	605	755	840	0.215	0.170	0.135	0.132	0.125
	1 day moist; remainder air of laboratory	315	395	470	605	755	0.206	0.156	0.116	0.138	0.185
	7 days moist; remainder air of laboratory	....	....	515	635	800	....	....	0.108	0.117	0.164
	28 days moist; remainder air of laboratory	....	....	....	615	800	....	....	....	0.101	0.135
Elgin Sand and Chicago Limestone	Moist room until test	335	510	705	835	945	0.200	0.175	0.149	0.128	0.118
	1 day moist; remainder air of laboratory	335	410	545	605	685	0.192	0.145	0.132	0.136	0.162
	7 days moist; remainder air of laboratory	....	....	565	615	760	....	....	0.116	0.112	0.141
	28 days moist; remainder air of laboratory	....	....	....	625	815	....	....	....	0.100	0.131
		TENSILE STRENGTH					TENSILE STRENGTH TO MODULUS OF RUPTURE				
Elgin Sand and Gravel	Moist room until test	175	240	360	415	525	0.52	0.54	0.60	0.55	0.62
	1 day moist; remainder air of laboratory	155	235	315	410	370	0.49	0.59	0.67	0.68	0.49
	7 days moist; remainder air of laboratory	....	....	380	470	470	....	....	0.74	0.74	0.59
	28 days moist; remainder air of laboratory	....	....	....	490	495	....	....	....	0.80	0.62
Elgin Sand and Chicago Limestone	Moist room until test	205	295	415	540	590	0.61	0.58	0.59	0.65	0.62
	1 day moist; remainder air of laboratory	190	275	375	445	380	0.57	0.67	0.69	0.74	0.56
	7 days moist; remainder air of laboratory	....	....	370	525	525	....	....	0.66	0.85	0.69
	28 days moist; remainder air of laboratory	....	....	....	550	570	....	....	....	0.88	0.70

those obtained with the other coarse aggregates, probably due to poorer bond between the mortar and the smooth, hard coarse aggregate particles in the case of the granite, and to lack of structural strength to resist tensile stress in the case of the sandstone; in the flexural and

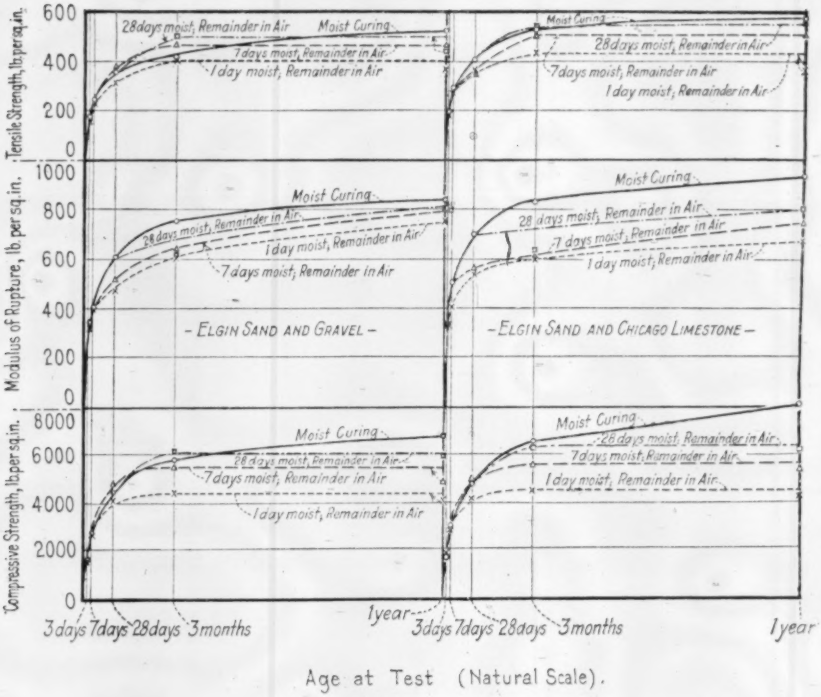

FIG. 8.—Compressive Strength, Modulus of Rupture and Tensile Strength of Concrete for Different Curing Conditions.

Data from Table VII.
Compression tests of 6 by 12-in. cylinders.
Flexure tests of 7 by 10 by 38-in. beams.
Tension tests of 6 by 18-in. cylinders.
Mix 1:4 by volume.
Aggregate: Fine; sand from Elgin, Ill.
Coarse; Elgin gravel and Chicago limestone.
Consistency constant; flow 200 per cent ± 10 per cent.

tension tests failure occurred through the soft sandstone particles. These results show that an aggregate which produces a concrete of high compressive strength may not necessarily give high strengths in tension or flexure.

Included in Table VI also are the results of tension tests on concrete in which the coarse aggregates were siliceous gravel, dolomitic limestone and trap rock. Concrete from these three coarse aggregates was generally somewhat lower in tensile strength for the same mix and approximately the same water-cement ratio, than that from the other coarse aggregates tested. Due to lack of material, it was not possible to make compression and beam specimens from these aggregates.

Because of the differences in strengths shown by thea ggregates used in these tests it should not be inferred that one type of coarse aggregate will necessarily give better results in concrete than another. Other sources of a given type might show entirely different results, and, furthermore, differences in strength can generally be overcome by changing the proportions of the mix. A comparison of materials from different sources for a particular purpose should include a study of the changes in proportions necessary to produce the desired strength, as differences in cost of materials may offset differences in proportions, or *vice versa*.

Strength of Concrete Cured Under Different Conditions.—The results of compression, flexure and tension tests on concrete cured under different conditions are given in Table VII and Fig. 8. In these tests, the mix and consistency were maintained constant and the age at test ranged from 3 days to 1 year. Both Elgin gravel and Chicago limestone were used. The data for the different curing conditions are not directly comparable since the moist-cured specimens were tested damp while the specimens cured for a time in the moist room and then in air in the laboratory were either partially or well-dried when tested. The extent of the drying depended on the duration of air storage.

A comparison of the curves in Fig. 8 shows that the age-strength relations obtained with the limestone and gravel concretes are similar. The moist-cured specimens in all cases increased in strength with age. Compression and tension specimens cured in air in the laboratory, after 1, 7 and 28 days moist curing, showed increases in strength up to about the 3-month period and no increase thereafter; whereas for these methods of curing the modulus of rupture continued to increase with age. The longer the period of moist curing before removal to the air of laboratory, the higher were the strengths at the later ages.

When specimens were removed from the moist room at various ages and placed in air of laboratory, the compressive strengths were generally somewhat higher for a short period following removal than for similar moist-cured specimens. The apparently higher strengths in the compression tests, due to the lower moisture content of the specimens at time of test, have been shown by other tests in this and other laboratories. However, the flexural strengths were always lower for a short period following removal from moist curing than for similar moist-cured specimens. This is exactly the reverse of the results obtained in the compression tests. The data from the tension tests are not so well defined and are somewhat conflicting. In this connection, it is of interest to note that earlier tests made in this laboratory and confirmed by tests now under way show that beams soaked before test give a higher modulus of rupture than when tested dry.

TABLE VIII.—TENSILE STRENGTH OF CONCRETE CYLINDERS 6-IN. IN DIAMETER OF VARIOUS LENGTHS.

Tension tests of concrete cylinders, 6-in. in diameter, 12, 18, 24 and 30-in. long. One specimen of each length made from each batch.
Mix 1:4 by volume.
Aggregate: Elgin sand and gravel graded 0 to 1½ in. (Fineness Modulus, 5.50).
Age at test, 28 days.
Each value is the average of 5 tests made on different days unless otherwise indicated.
For further details of tests see notes accompanying Table II.

Water-Cement Ratio		Flow, per cent	Weight of Concrete, lb. per cu. ft.	Cement, sacks per cu. yd.	Tensile Strength, lb. per sq. in.			
Nominal	Net				6 by 12-in. Cylinder	6 by 18-in. Cylinder	6 by 24-in. Cylinder	6 by 30-in. Cylinder
0.82	0.76	208	152	6.3	345	355[a]	330	345

[a] Average of 15 tests.

Effect of Length and Size of Specimen on Tensile Strength.—Tension tests were made on cylinders 2, 6 and 10 in. in diameter and 3 diameters long, and on 6-in. cylinders 2 to 5 diameters long. The results of these tests, given in Tables IV and VIII, show that the tensile strength of the concrete was not appreciably affected by either the length or size of cylinder. The results obtained with the 2 by 6-in. cylinders were somewhat erratic due, no doubt, to the lack of sensitiveness of the testing machine at the relatively low loads sustained by this size of specimen. The strengths of the 10 by 30-in. cylinders compared very favorably with those of the 6 by 18-in. cylinders.

A study of the location, with respect to the grips, of the section where failure occurred in the tension tests showed that, in general, about the same number of failures occurred at the upper as at the lower grip. About 28 per cent of the 3-day specimens and about 40 per cent of the specimens tested at the later ages failed near the middle.

Uniformity of Tests.—In recent years considerable interest has been shown by engineers in the development of a type of test convenient for field control of concrete quality. On road construction the beam test is becoming more generally used than the compression test and is believed by some engineers to give more uniform results. A study was made of the uniformity of strengths for the different types and conditions of test employed in this investigation. In this study the uniformity was judged by the mean variations—the mean of the variations of the individual strengths from their average expressed as a percentage of the average.

On the whole, there was little difference in the uniformity of the strengths obtained in the three types of test. The compressive and flexural strengths showed about the same degree of uniformity, which in a few of the tests was somewhat better than that of the tensile strengths. In general the mean variations for each of the three types of test for the other variables studied were:

1. Somewhat higher at the early ages than at the later;
2. Appreciably higher for the dry, non-plastic concretes than for the plastic, although the mean variations for the very wet concretes were also relatively high;
3. Somewhat higher for the lean than for the rich mixes;
4. Essentially the same for workable mixes in the tests where the size and grading of the aggregate was varied (average mean variation for each type of test about 5.0 per cent); and
5. Higher for the tension tests of 2 by 6-in. and 10 by 30-in. cylinders than for the 6 by 18-in. cylinders.

The higher variations obtained in the tension tests of the 2 by 6-in. cylinders were due mainly to the fact that the sensitiveness of the testing machine would not permit of very accurate tests at the low loads sustained by these cylinders. The 10 by 30-in. cylinder was rather bulky and unwieldy and test conditions were not quite as favorable for this as for the 6 by 18-in. cylinder.

CONCLUSIONS

The principal conclusions from the tests are as follows:

1. The relations between the water-cement ratio and the modulus of rupture and tensile strength of plastic, workable concrete mixes are similar to the water-cement ratio - strength relations for com-

pression tests, showing that for given materials and methods of manipulation, the water-cement ratio is a governing factor in the strength of concrete whether it be tested in compression, flexure or tension. Increasing the amount of mixing water reduced the strength for all three types of test, the effect being relatively greater on compressive strength than on either the modulus of rupture or tensile strength,

2. Typical age-strength relations were obtained with each of the three types of test. The strengths of moist-cured concrete in all cases increased with age. Compression and tension specimens cured in air of laboratory after 1, 7 and 28 days moist curing, increased in strength up to about the 3-month period and showed little or no increase thereafter, whereas for these methods of curing the modulus of rupture continued to increase with age. The longer the period of moist curing before removal to the air of laboratory, the higher were the strengths at the later ages.

3. A feature of the tests was the advantage shown, by all three types of test, in the use of a low water-cement ratio in the production of high early strength concrete. Reducing the water-cement ratio not only increased the strengths at the later periods, but also greatly increased the early strength so that it was a much higher precentage of the later strength.

4. The tests show that such factors as size and grading of the aggregate affect the compressive and tensile strengths and the modulus of rupture of concrete of a given mix and consistency, only as they affect the amount of water required to produce the desired consistency. The strength increased with the size of the aggregate and also with increase in the proportion of coarse to fine so long as the proportions of coarse to fine did not result in an undersanded mix. For increase in the coarseness of the grading into the range of undersanded mixtures, the compressive and tensile strengths dropped off sharply, whereas the modulus of rupture continued to increase. In view of these results it would seem that flexure tests for field control of concrete quality should be used with some caution, particularly with coarsely graded aggregates, since tests of undersanded or honeycombed and porous concrete might through the higher flexural strengths indicate a better quality of concrete than was actually being obtained.

5. There were well-defined relationships between the compressive strength, modulus of rupture and tensile strength of moist-cured concrete made from Elgin sand and gravel. The relation between the modulus of rupture and the tensile strength was practically con-

stant. The relations of these strengths to the compressive strength were not constant but were nearly so at the higher strengths. The modulus of rupture of concrete having a compressive strength of from 2000 to 5000 lb. per sq. in. ranged from about one-fifth to one-seventh of the compressive strength and the tensile strength from one-tenth to one-twelfth of the compressive. This shows that the tensile strength was about 0.5 to 0.6 of the modulus of rupture.

6. Similar well-defined relations between the three types of test were found for the few coarse aggregates other than Elgin gravel included in the investigation. Except for the soft sandstone, the relations for the other coarse aggregates were very nearly the same as for Elgin gravel. In the case of the sandstone, the flexural and tensile strengths did not show increases corresponding to increase in compressive strength for compressive strengths above 3000 lb. per sq. in. These differences show that the flexural and tensile strengths are more responsive to the physical characteristics of the aggregates than is the compressive strength.

7. The tensile strength was not appreciably affected by either the length or size of the cylinder.

8. In general, there was about the same degree of uniformity in the strengths obtained with the three types of test.

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DISCUSSION

Mr. Kellermann.

MR. W. F. KELLERMANN[1] (*presented in written form*).—As has been brought out in the paper by Messrs. Gonnerman and Shuman, there is very little published information on the various factors affecting the tensile and flexural strength of plain concrete. Within recent years considerable attention has been given to the flexural strength as applied to concrete for pavements and the field testing of beams is increasing in popularity, due to the ease with which the test can be made on the job. This is not true of the tension test, however, as the methods for testing now in use require laboratory apparatus. If definite ratios existed between tension, compression and cross-bending, the task of designing would be simplified, because the other strength properties of the concrete could be approximated for a given set of conditions by merely running one type of strength test. The authors have taken a step forward along this line by establishing relationships between the results of the three types of strength test for a given set of conditions by making the different tests on specimens made from the same batches of concrete.

Some work has also been done on tension and cross-bending in the concrete laboratory of the U. S. Bureau of Public Roads, and the results obtained furnish interesting comparisons with those presented by Messrs. Gonnerman and Shuman. In our work the beams were 6 by 6 by 30 in. and were tested as cantilevers. This method gave somewhat higher results than those obtained with the method of loading at the third points on the same size beam. The tension test cylinders were 6 by 21 in. and were filled in three layers. Compression results were obtained by capping the broken tension specimens and storing them until the next period for testing. They were not all of the same height and the strengths were corrected to a specimen whose height was twice the diameter.

For breaking the tension specimens, a set of grips similar to those used in the Portland Cement Association laboratory were employed, about 5 in. of the specimen being held in each grip. By using a 21-in. cylinder, it was usually possible to obtain two breaks on the same specimen. This, of course, made a smaller distance between grips on the second break than on the first, and is partly responsible for the fact that in most cases the second break was about 10 per cent higher

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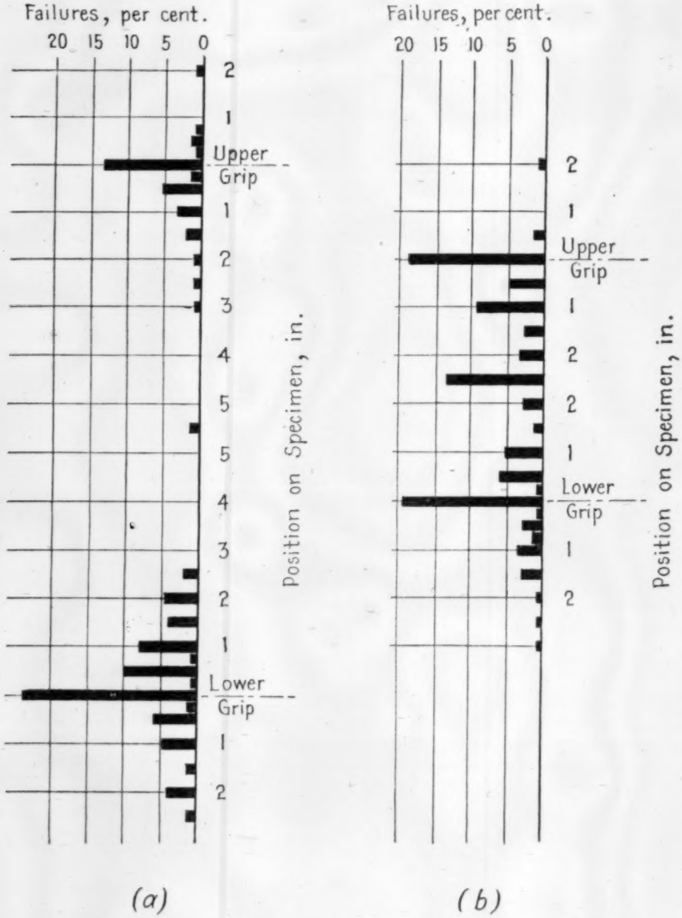
[1] Assistant Materials Engineer, U. S. Bureau of Public Roads, Washington, D. C.

Mr. Kellermann. than the first. That the distance between grips affects the position of the break may readily be seen by reference to the accompanying Fig. 1, which shows that the percentage of failures near the center of the specimen increased as the distance between grips decreased. No doubt the stresses caused by tightening the grips was responsible for such a large percentage of the breaks occurring at or very close to one of the grips.



(*a*) Position of First Breaks, 157 Tests. (*b*) Position of Second Breaks, 153 Tests.

FIG. 1.—Showing Relation Between the Percentage of Failures and the Distance from Grips.

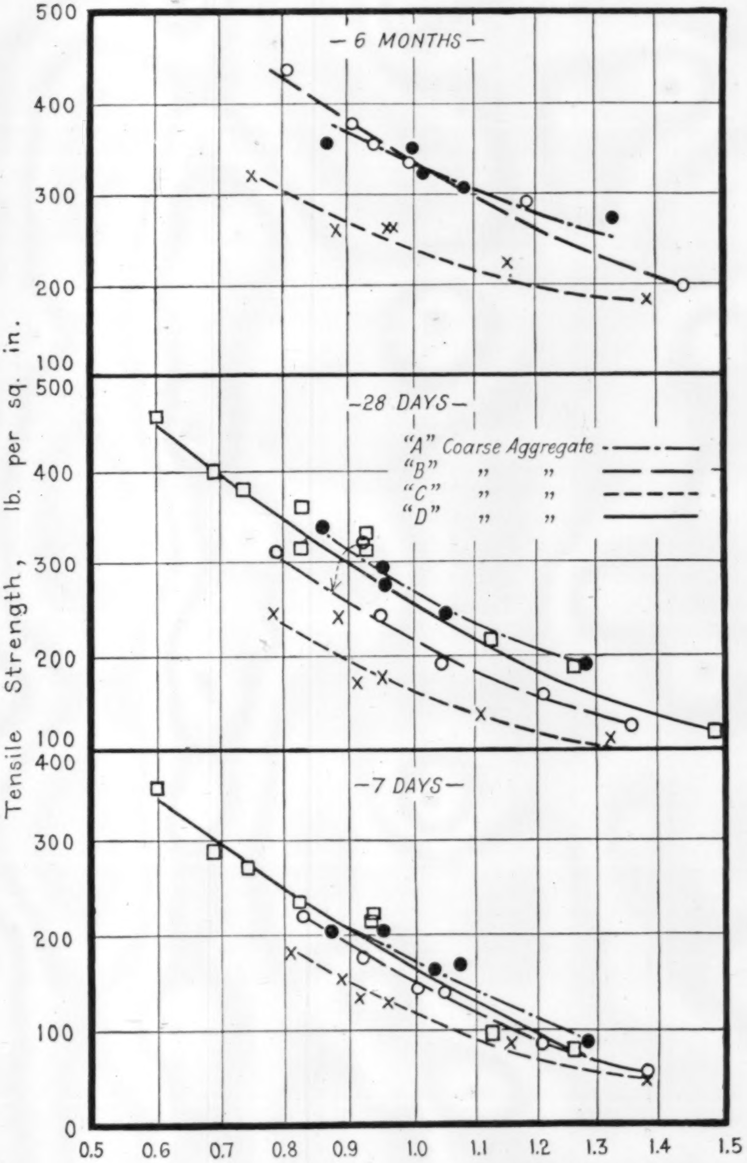


FIG. 2.—Water-cement Ratio - Tensile Strength Curves for Concrete Using Four Different Coarse Aggregates.

Mr. Kellermann.

In Fig. 2 are plotted water-cement ratio - tensile strength curves for four different coarse aggregates. For aggregates A, B and C, the same sand and cement were used, while the points for aggregate D were plotted from the data presented by Messrs. Gonnerman and Shuman. Although for some of the curves the points are few in number, the general trend for each aggregate gives unmistakable evidence that the coarse aggregate is an important factor in the resistance of concrete to pure tension. From this curve it will be noted that aggregate C gave the lowest strengths and that the difference was greater at 28 days and 6 months than at 7 days. No doubt the bond between aggregate and mortar and the strength of the aggregate itself are factors which influence the tensile strength to a marked degree.

Ratios found between the tensile and compressive strengths for three aggregates are given in the following table:

Compressive Strength, lb. per sq. in.			Ratio, Tensile Strength to Compressive Strength		
Aggregate A	Aggregate B	Aggregate C	Aggregate A	Aggregate B	Aggregate C
AGE 6 MONTHS					
4450	3720	3790	0.081	0.117	0.085
3780	3580	3280	0.094	0.109	0.078
3790	3440	2920	0.085	0.112	0.091
3360	3200	2650	0.092	0.112	0.101
2650	2360	2210	0.103	0.142	0.103
....	2160	1830	.....	0.135	0.101
....	1730	....	.....	0.116	.....
....	1440	....	.....	0.149	.....

It is quite evident from the above table that aggregate B gave consistently higher ratios than either of the other two aggregates. The general tendency was for the ratio to decrease with an increase in compressive strength, which is in general agreement with that found by Messrs. Gonnerman and Shuman.

Although the number of flexure specimens tested in connection with this work was too small to permit of any definite statement as to the ratio of tensile strength to modulus of rupture, there are indications that this ratio increases somewhat with increase in strength of the concrete.

Experience with different coarse aggregates in our concrete laboratory give strong indications that the compressive strength is not exactly indicative of the tensile and flexural strengths and that ratios established for any one aggregate should not be used indiscriminately.

Mr. Davis.

MR. R. E. DAVIS[1] (*presented in written form*).—Referring to the paper by Messrs. Gonnerman and Shuman, it has seemed to me for

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[1] Professor of Civil Engineering, University of California, Berkeley, Calif.

Mr. Davis. some time that the compressive strength of concrete ought not to be the sole criterion to its value as a material of construction. The thoroughness with which the problem of determining the relations that exist between strengths in compression, tension, and flexure was attacked leaves little to be desired.

The manner of gripping the tension specimen perhaps is worthy of remark. Obviously the action of clamping the ends of the specimens in the grips produces radial compression beneath the grips, and it might be expected that normally failure in tension would occur at the grips, the value being less than the true tensile stress that would be developed under the action of simple tensile stress unaccompanied by radial compression. The statement is made that 28 per cent of the 3-day specimens and 40 per cent of those of greater age failed near the middle. It would be of interest to know if those which failed in the grips consistently exhibited lower tensile strengths than corresponding specimens which broke near the middle. Some years ago at the University of California we experimented with several types of tension specimens in connection with a series of tests to determine the relative efficiency of several methods of treatment of construction joints. The specimen which we finally used was 6 in. in diameter by 24 in. in length, in either end of which were cast rods of varying lengths. The outer ends of these rods projected beyond the ends of the specimen and were threaded. By means of the rods a steel plate was secured to each end of the specimen. The tension was applied through a bar fixed at the center of the end plate. The function of the rods was to transmit the applied tension from the end plate to the concrete through bond. By varying the lengths of the imbedded rods the stress transition was made gradually.

In 1926, there was begun in the Materials Testing Laboratory of the University of California extensive tests on cement mortars. Among other things studies were made to determine the relation between strengths in compression, tension, and flexure. For all of the tests a single brand of portland cement and Sacramento River sand having a fineness modulus of 2.0 were used. The mix varied from 1:3 to 1:5 by volume. Various plasticities were secured through the use of various proportions of lime and clay. On the one extreme, the mortar contained no admixture; on the other, the mortar contained by volume as much lime as cement. Specimens were cured in air at 70° F. and 50 per cent relative humidity. The compression specimens were 2 by 4-in. cylinders; the tension specimens were standard briquets; and the flexure specimens were 2 by 2 by 12-in. bars tested on a 10-in. span with load centrally applied. In all, eleven

Mr. Davis. mortars were tested at ages ranging from 1 day to 6 months. The water-cement ratio varied but the consistency was constant for all mortars, being such as to produce a slump of $3\frac{1}{2}$ in.

In the accompanying Fig. 3 are diagrams similar to those of Fig. 6 of the paper. The ordinate of each plotted point of the lower group is the ratio of the average tensile to the average compressive stress of corresponding groups of specimens. Similarly, the ordinate of each plotted point of the upper group represents the ratio of average modulus of rupture to compressive stress of corresponding groups of specimens. Through the plotted points are drawn mean curves.

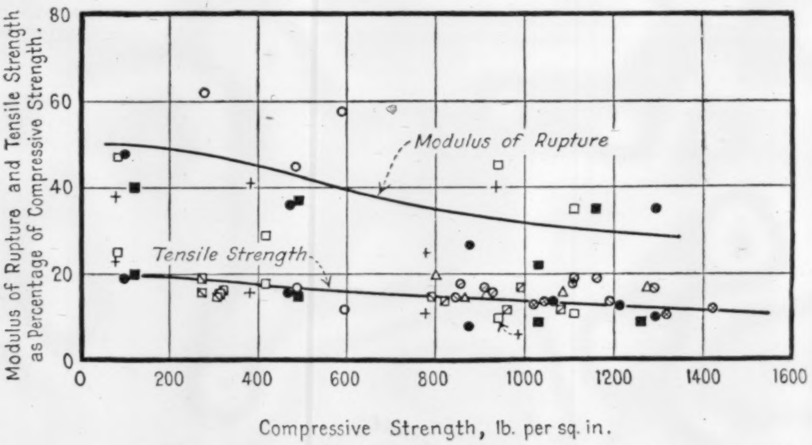

FIG. 3.—Showing Ratio of Compressive Strength to Modulus of Rupture and to Tensile Strength of Concrete.

On comparing the curves of the accompanying Fig. 3 with those of Fig. 6 of the paper, the following observations may be made:

1. For the mortars there is a consistent relation between tensile strength and compressive strength regardless of the amount or character of the material forming the admixture, and regardless of the age of the specimen at the time of test. This relation is not constant but varies inversely as the magnitude of the compressive strength, the tensile strength being about 20 per cent of the compressive strength when the latter is 50 lb. per sq. in. and 11 per cent of the compressive strength when the latter is 1400 lb. per sq. in.

2. For the mortars there is a less well defined but nevertheless a definite relation between modulus of rupture and compressive strength. The amount and character of the material forming the admixture, whether lime or clay, does not appear to influence this relation to

Mr. Davis.

any marked degree; nor does the age of specimen. The ratio of modulus of rupture to compressive strength varies considerably with the compressive strength, the modulus of rupture being about 50 per cent of the compressive strength when the latter is 100 lb. per sq. in. and roughly 30 per cent of the compressive strength when the latter is 1400 lb. per sq. in.

3. For a given compressive strength, the ratio of tensile to compressive strength is higher for the mortars (Fig. 3) than for the concretes (Fig. 6). For the higher values of compressive strength this difference amounts to 5 per cent.

4. For a given compressive strength, the ratio of modulus of rupture to compressive strength is considerably higher for the mortars (Fig. 3) than for the concretes (Fig. 6). Thus at 1400 lb. per sq. in. it is 7 per cent higher for the mortars and at 500 lb. per sq. in. it is 18 per cent higher.

While on this general subject of strength tests it is perhaps appropriate to remark that between laboratories and between tests in the same laboratory there are sometimes inconsistencies which are difficult of explanation. In a series of tests to determine variation in compressive strength, Poisson's ratio and modulus of elasticity as affected by age, it was found that groups of tests gave results which appeared consistently too high or too low. The moisture content was maintained practically constant by the method of storage but the temperature at time of test varied with the season of the year. In an endeavor to determine whether or not temperature had any effect on strength within the normal range of atmospheric temperatures, a preliminary series of tests has been recently completed on watersoaked concretes and mortars tested at the ages of 7, 28, and 135 days. The temperature at time of test varied from 36° F. to 140° F. In every case the results indicated a marked decrease in strength with increase in temperature.

While the experiments are not sufficiently comprehensive to make the data of any value in estimating the effect of temperature, yet the results are sufficiently conclusive to leave no doubt but that the strength of portland-cement mortar or concrete when watersoaked is affected appreciably by the temperature at time of test. But this is, of course, no criterion to the effect on the strength of concrete or mortar for which the moisture content is low, as when allowed to dry over a long period of time.

Mr. Brickett.

MR. E. M. BRICKETT.[1]—In Fig. 5 of the paper it is noticed that the tensile strength and the compressive strength of the concrete fall

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[1] Engineer of Tests, Research Division, Lehigh Portland Cement Co., Allentown, Pa.

Mr. Brickett. off after the fineness modulus has passed a value of possibly 5.5, and yet the modulus of rupture continues to increase. I wonder whether the authors have any explanation for this apparently unreasonable phenomenon. I wonder whether the apparently peculiar relations are due to the size and shape of the several types of specimen. Six-inch cylinders, 12 in. in length, were used for the compression values, 18 in. in length for the tension values. For the transverse test 7 by 10-in. beams were used. The cylinders were presumably poured on end and the beams on their sides. It is easily conceivable that the concrete is so harsh after this critical fineness modulus is passed that the concrete in the 6-in. cylinders, particularly in the longer ones, could not be thoroughly and uniformly compacted so that the strengths obtained are not true, but are lowered by localized faults in the concrete. The beams, on the other hand, are of larger section and, since they are poured on their sides, the concrete is accessible for thorough tamping with the result that a dense and uniform concrete is obtained and the modulus of rupture shows a consistent increase.

Mr. Gonnerman. MR. H. F. GONNERMAN.[1]—I think that is partly but not wholly the answer to the marked falling off in strength of the compression and tension cylinders as compared with the modulus of rupture of the beams. In molding the cylinders, the concrete was placed in layers and rodded in the usual manner. In molding the beams, the concrete was placed in two layers; the beam form rested flatwise on the floor of the laboratory and each layer was rodded about 75 times and the sides of the beam were spaded with a trowel. This treatment may have tended to compact the harsh working mixtures to a greater extent in the beams than in the cylinders.

We repeated these tests and got essentially the same indications as shown in this diagram although there was in some cases a dropping off in modulus of rupture at values of fineness moduli of 6.25 and 6.50. This falling off in modulus of rupture was not as clearly defined as in the case of the compression tests which gave identical indications as the earlier series. When the percentage of sand is as low as 14 per cent by weight of mixed aggregate of the grading used in these tests, there is not much sand in the mixture, and in a 1:4 mix by volume a harsh-working concrete is obtained.

Regarding the remarks by Mr. Davis, there is a combined stress effect near the grips, and at the early ages a large number of breaks occurred there. At the later ages more of the breaks occurred at or near the middle of the specimen. We do not know the exact effect

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[1] Manager, Research Laboratory, Portland Cement Association, Chicago, Ill.

Mr. Gonnerman.

of these combined stresses but judging from the results of tension tests by another method, we do not believe that they had much influence on the strengths obtained.

We tried the method of making tension tests described by Mr. Davis, except that where he used six bars we used three embedded in each end of the cylinders. These bars were then attached to plates and long bars carried the load from the plates to the heads of the testing machine. The results of tests on 1:4 concrete at 7 and 28 days show about the same strengths and degree of uniformity as was obtained with the friction grips. The specimens made with embedded rods were more difficult to make but were somewhat easier to test.

Mr. Davis' tests on mortars seem to confirm those described in our paper. It should be noted, however, that his specimens gave lower compressive strengths (100 to 1400 lb. per sq in.), consequently we would expect that he would obtain higher ratios of tension to compression as the curve showing the relation between these ratios and compressive strength rises at the lower values of compressive strength.

Mr. Goldbeck.

MR. A. T. GOLDBECK.[1]—The paper by Messrs. Gonnerman and Shuman is a valuable contribution to the literature of a phase of concrete testing about which comparatively little has thus far been published. As would be expected, the general relation of strength to water-cement ratio for any one set of conditions holds for tension and flexure tests as well as for compression, although quantitatively these relations are not identical. One of the most interesting phases of the investigation is that dealing with the effect of the coarse aggregates on the modulus of rupture and tensile resistance of concrete. It is quite apparent from the test results, which are confirmed by other series of a similar nature, that the coarse aggregate has far more influence on the beam strength and tensile resistance of concrete than on the compressive resistance. For concrete used in concrete highway construction this is highly significant, for the resistance of concrete highways is dependent not on compressive resistance but on tensile resistance and on beam strength. It is quite apparent that very soft aggregates or aggregates which for some reason do not provide a good bond for the mortar may give high compressive strength, but relatively low modulus of rupture. It seems important, then, that the cross-bending test be given far more weight in judging the quality of road concrete than the compression test. I should like to ask Mr. Gonnerman a question in connection with his Table VI and his Fig. 7. On the bottom of page 544 it states, "In the curves of Fig. 7 these differences appear somewhat less because the

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[1] Director, Bureau of Engineering, National Crushed Stone Association, Washington, D. C.

Mr. Goldbeck. dash-line curves of Fig. 7, representing the gravel concrete, cover a wider range of conditions." It is not quite clear just what is meant by "a wider range of conditions."

Mr. Gonnerman. MR. GONNERMAN.—Table VI gives a comparison of the concrete strengths for the three types of specimens made from Elgin gravel, Chicago limestone, Chicago blast-furnace slag, Wisconsin granite and Minnesota sandstone as coarse aggregate. The mix, fine aggregate, grading of mixed aggregate, and the flow were kept constant. Five specimens of a kind were tested at each age. A comparison of the strengths of the different concretes in this table shows that from about 10 to 20 per cent higher values were obtained for the Chicago limestone than for the Elgin sand and gravel. Instead of comparing the curves for each type of aggregate directly, we compared them with the more general curves for Elgin sand and gravel from Fig. 6 which cover the same range of ages but a much wider range in consistency, grading and size of aggregate and mix; all of these factors were not varied in the tests reported in Table VI.

Mr. Goldbeck. MR. GOLDBECK.—That is a point I could not understand, why concrete of given conditions should be compared with concrete having a wide range of conditions.

Mr. Gonnerman. MR. GONNERMAN.—The comparison could have been made under identical conditions. We thought it would be better to use the curves based on the widest possible range of conditions in making comparisons between the Elgin gravel and the other coarse aggregates. Direct comparisons of the results of tests in Table VI for each of the coarse aggregates are given in some detail in the text.

Mr. Mattimore. MR. H. S. MATTIMORE.[1]—The data in Messrs. Gonnerman and Shuman's paper is quite interesting at this time, due to the renewed interest in flexure and tension tests of concrete. Figure 5 of the paper, showing the comparative modulus of rupture and compressive strength as affected by the aggregates, checks data in a report on a series of tests published by me last year.[2]

My conclusion was that "Mixes with high sand content, due to the plastic character, give a more uniform specimen molded in a 6 by 12-in. cylindrical mold and up to a certain point, where the mortar is not too lean, have a tendency to give higher compressive strengths. As the coarse aggregate content increases it is more difficult to mold the specimen, and the compression test either shows loss or at least no appreciable gain, while at the same time the transverse test reflects the increase in strength due to larger stone content and richer mortars."

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[1] Engineer of Materials, Pennsylvania State Highway Department, Harrisburg, Pa.

[2] "Transverse Tests as a Criterion of the Quality of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 383 (1927).

Mr. Mattimore. In conclusion 4, the authors state, "For increase in the coarseness of the grading into the range of undersanded mixtures, the compressive and tensile strengths dropped off sharply, whereas the modulus of rupture continued to increase. In view of these results it would seem that flexure tests for field control of concrete quality should be used with some caution, particularly with coarsely graded aggregates, since tests of undersanded or honey-combed and porous concrete might through the higher flexural strengths indicate a better quality of concrete than was actually being obtained." This conclusion can be considered in a class of assumption, from the standpoint that the tabulated test results which I published last year were tests on concrete which was actually used in pavement slabs which had sufficient workability to give a good finish and could not be considered in either the honey-combed or porous class of concrete.

The statement in conclusion 4, that "the higher flexural strengths indicate a better quality of concrete than was actually being obtained," must be predicated entirely on the idea that the compression test of concrete is the only true value of quality. There is no reason to believe that this is a fact, and, further, there are no data available at the present time indicating that there is a relation between compressive strength and other qualities of concrete.

Mr. Brickett. MR. BRICKETT.—This is rather an important matter. Highway officials are now coming to favor the flexure test for concrete, mainly, I believe, because the test is so easy to make. We should like to know whether or not the flexure test is bringing out some properties in concrete, possibly desirable properties, that the compression test does not show. The results of this paper are somewhat misleading in this respect. Mr. Gonnerman admits the difficulty of compacting the concrete in the 6-in. cylinders as compared with the beams. The placing of concrete is just as important in determining quality of concrete as is water-cement ratio or any other factor. Figure 5 of the paper compares the tensile, compressive and transverse strengths of concrete, but it is not the same concrete in each case because of the different degree of compacting. Figure 5 is more valuable in showing clearly the limitations in this series of tests than in showing absolute strength relations up to the maximum fineness modulus.

Mr. Gonnerman.. MR. GONNERMAN.—The same concrete was used in the three types of specimen since they were made from the same batch. However, as stated previously, there may have been some differences in the placing of the concrete in the three types. In the modulus of rupture curve, the last two mixes were harsh-working, still there was not the marked dropping off in this curve that occurred in the curves

Mr. Gonnerman. for compressive and tensile strength. This does not necessarily mean that the beam test is not a useful one in judging the quality of concrete but it does indicate, I think, that when harsh mixes are used care must be taken in interpreting the results of flexure tests as there is a tendency to place harsh-working concrete better in the test beam than in a pavement, for example, and thus the test results may not truly indicate the character of the concrete in the pavement. Furthermore, in the compression or tension specimens the entire cross-section is uniformly stressed whereas with the beam the maximum stress exists only at the outermost fibers where, on account of the method of placing, the concrete is probably of a somewhat better quality than that in the body of the specimen. The maximum stress in the usual method of testing beams occurs over a comparatively short section of the specimen and honeycombing in the interior is not as likely to cause as rapid a dropping off in strength as is the case with the other two types of specimen.

THE EFFECT OF SEVERAL MECHANICAL FEATURES OF TESTING ON THE DETERMINATION OF THE FLEXURAL STRENGTH OF PLAIN CONCRETE

By T. F. Willis[1] and F. N. Wray[1]

Synopsis

The paper points out the diversity in the generally accepted methods for making flexural tests of concrete. The relation between the results obtained by several of these different methods and the effect of certain features, such as length of span, type of bearings, and rate of application of stress, are shown.

Results obtained with the different methods vary widely, and for certain methods the length of span and type of bearings affect the results considerably, indicating the need for standardization of this test.

With the adoption of flexural resistance of concrete as a basis for rational and economical design of concrete slabs for pavements, testing laboratories throughout the country have been called upon to measure this quality of concrete. Ease and simplicity of operation, together with small cost and mobility of apparatus, have contributed to the popularity of the flexural test as a measure of quality of pavement concrete.

None of the organizations responsible for the promulgation of standards has adopted or recommended a standard method of making this test.[2] As a result, each laboratory called upon to perform the test has devised its own apparatus and methods. A great diversity of apparatus and methods has resulted, which is particularly noticeable in the testing that has been done in the field. With the idea of determining the effects of some of the features embodied in various machines and of correlating test results obtained on various types of machines, the investigation described below was undertaken.

Scope of Investigation

The investigation consisted of the testing of 115 plain concrete beams and 30 cylinders made and cured as nearly alike as possible.

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[1] Bureau of Materials, Missouri State Highway Commission, Jefferson City, Mo.

[2] Subsequent to the writing of this paper, the American Association of State Highway Officials has appointed a committee, of which F. V. Reagel, Engineer of Materials, Missouri State Highway Commission, is chairman, to work on this problem.

Three different types of testing machines were used; two of these were flexural testing machines of special design, and the third was a 200,000-lb. Riehle machine.

Special attention was given to the following:

1. The effect of the distances (*a* and *b* in Fig. 1) between points of loading and reaction, using fixed knife-edge bearings, on the breaking load and calculated modulus of rupture obtained with Machine No. 1.

2. The effect of using roller bearings instead of knife-edge bearings on results obtained with Machine No. 1.

TABLE I.—EXPERIMENTAL RESULTS USING VARIOUS METHODS OF FLEXURE TESTING.

Flexural Tests on 6 x 6 x 36-in. beams.
Compression Tests on 6 x 12-in. cylinders.
Mix 1:2:3½ (by dry, rodded volumes).
Effective water-cement ratio 0.68.
Consistency 1-in. slump.

Cured: Wet burlap 24 hours, Moist room 27 days
Temperature 75 to 82° F., Relative Humidity 95 to 100 per cent.
Tested damp at 28 days.
Each value is the average of 6 breaks.

	Modulus of Rupture, lb. per sq. in.											Compressive Strength, lb. per sq. in.
	Method I	Method II	Method III	Method IV	Method V	Method VI	Method VII	Method VIII	Method IX	Method X	Method XI	
Tested on Machine	No. 1	No. 1	No. 1	No. 1	No. 1	No. 2	No. 2	No. 2	Riehle	Riehle	Riehle	Riehle
Type of Bearings	Rigid	Rigid	Rigid	Roller	Roller	...	....	...	Roller	Roller	Roller	....
Distribution of Load and Reaction	*a*, 6 in. *b*, 6 in.	*a*, 10 in. *b*, 10 in.	*a*, 20 in. *b*, 10 in.	*a*, 6 in. *b*, 6 in.	*a*, 10 in. *b*, 10 in.	...	...	..	30-in. span at third points	30-in. span at mid-point	12-in. span at mid-point	....
Rate of Application of Stress, lb. per sq. in. per second	5 to 6	5 to 6	5 to 6	5 to 6	5 to 6	2.3	5.6	8.1	5 to 6	6 to 7	5 to 6	60
Series No. 1	725	575	495	...	...	460	470	455	435	...	...	3300
Series No. 2	720	530	470	...	...	460	465	440	415	...	...	3650
Series No. 3	645	485	465	...	...	420	410	415	410	...	...	3470
Series No. 4	655	505	...	515	505	...	450	...	440	500	530	3980
Series No. 5	700	525	...	495	520	...	455	...	435	480	515	3510
Average	689	524	477	505	512	447	450	437	427	490	522	3582
Per cent of Method IX	161	123	112	118	120	105	105	102	100	115	122	....

3. The effect of the rate of application of stress on test results obtained with Machine No. 2.

4. The comparison between results obtained with 30 and 12-in. spans loaded at centers and supported on roller bearings in the Riehle machine.

5. A comparison of each of the preceding with the results obtained using a 30-in. span loaded at the one-third points in the Riehle machine.

6. The compressive strength of the concrete used.

The different methods of testing are designated I to XII as indicated in Table I.

APPARATUS

The machine shown in Fig. 1 (designated Machine No. 1) was designed by the authors and has been used by the Missouri State Highway Department for over a year. The test specimen rests on the plate *E* and the knife-edge bearing *C*. The rear end of the specimen is held down by means of the screw and bearing *Z*. The forward end of the specimen overhangs and the load is applied through a bearing *K*, which in some cases was a rigid knife-edge and in others a roller.

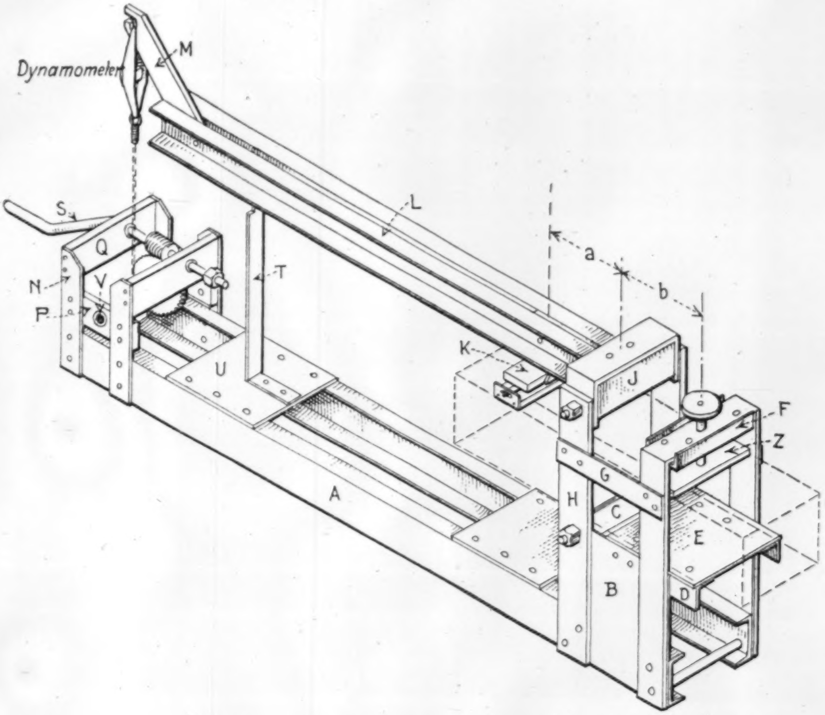

FIG. 1.—Testing Machine for Determining Flexural Strength of Plain Concrete.

The lever arm *L* is pivoted at *J* directly over the knife-edge *C*. The load is produced by deflecting the lever arm by means of the cable and windlass, and is measured by a dynamometer. While original in design, the methods of transmitting and applying the load to the test specimen are similar to those incorporated in the machines used by a number of other agencies.

The apparatus shown in Fig. 2 (designated Machine No. 2) was described by Clemmer and Burggraf.[1] The specimens are clamped as shown and a wooden lever arm fastened on the overhanging portion. The load is applied by running water or shot into a container hung on the end of the lever arm.

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[1] H. F. Clemmer and Fred Burggraf, "Transverse Testing of Concrete," *Proceedings*, Am. Concrete Inst., Vol. XXII, p. 304 (1926).

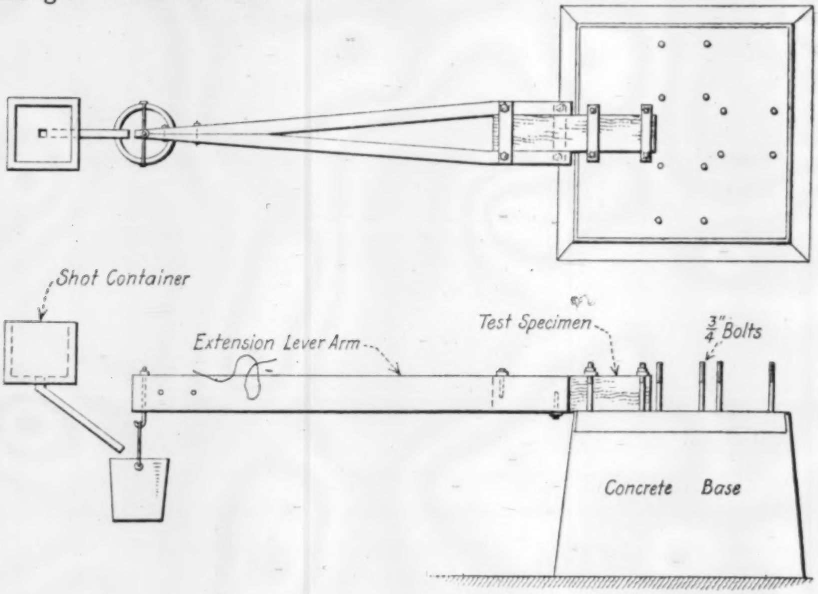
Shot Container
Extension Lever Arm
Test Specimen
$\frac{3}{4}$" Bolts
Concrete Base

FIG. 2.—Transverse Testing Apparatus.

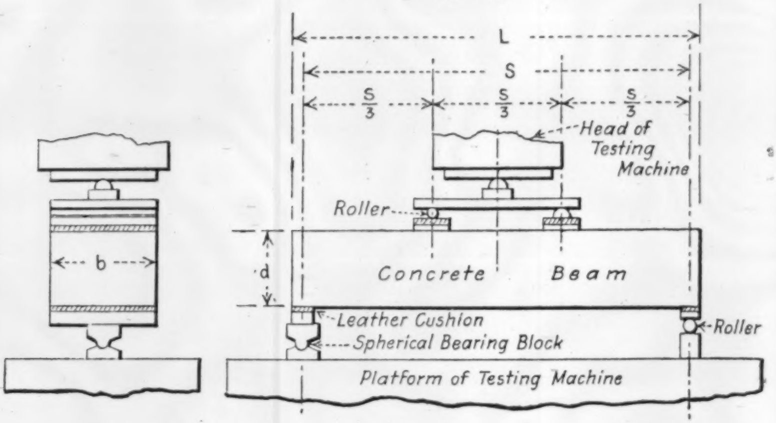
L
S
$\frac{S}{3}$
$\frac{S}{3}$
$\frac{S}{3}$
Head of Testing Machine
Roller
b
d
Concrete Beam
Leather Cushion
Spherical Bearing Block
Roller
Platform of Testing Machine

FIG. 3.—Sketch Showing Arrangement of Beams in Power Machine.

For making the one-third point loading tests, the set-up described by Duff A. Abrams,[1] shown in Fig. 3, was placed on the table of a 200,000-lb. Riehle machine.

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[1] Duff A. Abrams, "Flexural Strength of Plain Concrete," Structural Materials Research Laboratory, *Bulletin 11* (1922).

MATERIALS AND SPECIMENS

Materials.—The cement used in the experiments showed the following properties determined in accordance with the A.S.T.M. standard methods of testing:

Tensile Strength, lb. per sq. in.:	
7 day	260
28 day	385
Normal Consistency, per cent	22.2
Soundness	O.K.
Time of Set, hours and minutes:	
Initial	3:00
Final	5:30
Residue on No. 200 Sieve, per cent	9.9

The fine aggregate was a good quality of well-graded Missouri River quartz sand. The coarse aggregate was a good quality of well-graded Osage River chert gravel. The following properties of the fine and coarse aggregates were determined in accordance with the A.S.T.M. standard methods:

DESCRIPTION	FINE AGGREGATE Good quality well-graded Missouri River quartz sand	COARSE AGGREGATE Good quality well-graded ($1\frac{1}{2}$–$\frac{1}{4}$ in.) Osage River chert gravel
Weight, dry and rodded, lb. per cu. ft	110	90
Specific Gravity (apparent)	2.64	2.45
Void space in dry rodded material, per cent	33	41
Fineness Modulus	2.93	7.48
Absorption in 30 minutes, per cent by weight	0.0	2.0

Proportioning.—The proportions for all specimens were based on a 1:2:3.5 mix by dry rodded volumes of aggregates and the assumption that 1 cu. ft. of cement weighs 94 lb. The volume proportions were converted into weights and all measurements were made on an accurate balance.

Particular care was taken to see that the coarse aggregate gradation was the same in all batches of concrete. This was accomplished by screening the coarse aggregate into four sizes ($1\frac{1}{2}$ to 1 in.; 1 to $\frac{3}{4}$ in.; $\frac{3}{4}$ to $\frac{1}{2}$ in.; $\frac{1}{2}$ to $\frac{1}{4}$ in.) and using a fixed proportion by weight of each size in every batch of concrete. The batches were proportioned and mixed separately, and consisted of quantities of materials sufficient to yield approximately 0.85 cu. ft. of wet concrete. This amount was just slightly in excess of the requirement for one beam or three cylinder specimens.

Preparing Specimens.—All batches were mixed by hand on a metal surface, according to a fixed procedure. The cylinder specimens were made in accordance with the A.S.T.M. standard method. In molding the beams, the forms were filled one-half full, rodded fifty times, filled completely and rodded another fifty times, then finished with a wooden float and a trowel.

All specimens were cured under wet burlap for 24 hours and then placed in a moist room until tested at 28 days. The temperature of the room varied from 75 to 80° F., and the relative humidity from 95 to 100 per cent.

EXPERIMENTAL RESULTS

The flexural strength of the specimens was expressed as the modulus of rupture, calculated from the formula:

$$F = \frac{M}{Z}$$

where F = modulus of rupture in pounds per square inch, M = bending moment, in inch pounds, at failure, and Z = section modulus, in inches cubed.

Table I gives a summary of the experimental results obtained. The horizontal lines of values are designated by series numbers from 1 to 5. All the data in any one of these series were obtained from test specimens made on the same day. As many specimens were made in each series as were necessary to allow six breaks by each method of testing, the number of beams of one series required for a particular method of testing varying from two to six, depending on the length of span used.

It will be noted that for some of the methods of testing, specimens from all five series were tested; for others, specimens from the first three series were used; and for still others, specimens from the last two series were used. For those methods in which specimens from all five series were tested, the averages of the results obtained from the first three series are in fair accord with the averages of the last two series. Therefore, it is felt that the average of any one method is at least qualitatively comparable with that of any other method, regardless of the number of series from which the averages were obtained.

DISCUSSION OF RESULTS

Methods I, II and III show the effect of the distances (a and b in Fig. 1) between points of loading and reaction, using rigid knife-edge bearings. Method II gave results 31 per cent, and Method III 44 per cent, lower than Method I. From the results of these three methods it can be seen that as these distances increase, the modulus of rupture decreases.

Methods IV and V were similar to Methods I and II, respectively, with the exception that in the former, roller bearings were used instead of rigid knife-edge bearings. A greater load was required to cause the failure of beams tested in Methods I and II than those tested in Methods IV and V. Hence, the corresponding calculated moduli of rupture were higher. A probable explanation of this is as follows:

In Methods I and II, in which rigid bearings were used, the deformation of the fibers on the tension side of the test specimen, which should accompany an applied bending moment, was restrained by friction between the concrete specimen and the fixed knife-edge bearings, thus causing a transfer of a portion of the stress to the steel lever arm, *L*. In Methods IV and V, the roller bearings practically eliminated friction; hence there was little or no restraint of the deformation of the specimen, and the entire stress was concentrated in the specimen. It therefore follows that for test specimens having the same resistance to stress, those tested by Methods I and II would require greater loads to cause failure and would have higher calculated moduli of rupture than those tested by Method IV and V, respectively. Observation showed that there was movement of the tension side of the specimen in relation to the bearings when roller bearings were used, substantiating the assumption that fixed knife-edge bearings restrained the deformation of the fibers of the specimens.

It is interesting to note that while Method I (12-in. span) gave results 36 per cent higher than Method IV (same span), Method II (20-in. span) gave results only 2.5 per cent higher than Method V (also 20-in. span). This indicates that, as the distances between load and reactions were made greater, restraint of deformation of the fibers became less.

Methods VI, VII and VIII show the effect of the rate of application of stress on the results obtained with machine No. 2. In Method VII, 5.6 lb. per sq. in. per second (standard for the Missouri State Highway Laboratory) was the rate used. The rate used in Method VI was 60 per cent less than this, and that used in Method VIII 45 per cent greater. The results obtained by Methods VI and VII were practically identical, and those by Method VIII slightly lower. However, the results for all three methods were of the same order of magnitude, and it is considered that within the range of the experiment the rate of application of stress had no effect on the calculated moduli of rupture.

The results by Method IX were the lowest of all of the flexure test results. In this case the bending moment was constant and a maximum over the middle third of the beam Hence, the probability that the maximum moment would be applied at one of the weaker cross-sections of the beam was much greater in this case than in the other cases in which the bending moment was a maximum at only one cross-section.

Methods X and XI show the results obtained with the Riehle machine, using roller bearings, for two different span lengths. Method X, in which a 30-in. span was used, gave results 6 per cent lower than Method XI, in which a 12-in. span was used. These results show the same general relation to each other as those obtained by Methods I, II and III. The magnitude of the decrease is small, however, when compared with that shown in the case of Methods III and I, in which rigid bearings were used.

The relation between two or more different methods may be seen by comparing the figures in the last line of Table I, which express the average of each method as a percentage of the average of the results by Method IX.

Only two instances were found in which comparisons could be made between the data obtained in this investigation and those obtained by other investigators. During the discussion in connection with the Symposium on Field Control of the Quality of Concrete held at the 1927 annual meeting of this Society, L. W. Teller[1] gave results of two different methods of testing which are similar to Methods VII and X. His results for Method X are 12 per cent higher than those for Method VII, as compared with 10 per cent found in this investigation. In the same discussion, H. F. Gonnerman[2] gave results of two methods of testing which are similar to Methods IX and X. His results for Method X are 11 per cent higher than those for Method IX, as compared with 12 per cent found in this investigation. Both of these comparisons show close agreement.

The uniformity of the flexural test results varied for the different methods, having mean deviations from the averages as low as 1.5 per cent and as high as 7.7 per cent, but the majority of the methods fell within the range of 3.5 to 4.5 per cent. The data were not considered suitable for making close comparisons of the relative uniformity of the results of the different methods. However, there were indications that the use of roller bearings improved the uniformity of the tests. The average mean deviation for all the methods of flexural tests was 3.87 per cent of the weighted average of all of the flexural results, as compared with 5.42 per cent for the compression tests.

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[1] L. W. Teller, Discussion on Field Control of the Quality of Concrete, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 419 (1927).

[2] H. F. Gonnerman, Discussion on Field Control of the Quality of Concrete, *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 415 (1927).

The various methods of flexural testing gave moduli of rupture ranging from 11.9 to 19.2 per cent of the compressive strength.

It should be pointed out that there are conditions which may prevent these relations from being generally applicable. Strictly, they can be considered to apply only to the specimens tested. Different materials, mixes or specimen sizes may influence the results. However, it is felt that these same relations will hold, at least qualitatively, under most conditions.

CONCLUSIONS

The most important conclusions to be drawn from the investigation are:

In methods of flexure testing embodying the features of Machine No. 1 or the mid-point loading in the Riehle machine, the results are affected by both length of span and type of bearings. Increasing the length of span decreases the modulus of rupture. The effect of the length of span is greater when fixed knife-edge bearings are used than when roller bearings are used. When fixed knife-edge bearings are used instead of roller bearings, in conjunction with spans less than 20 in., the moduli of rupture are higher; but with longer spans the type of bearings has little or no effect.

For methods of testing embodying the features of the Clemmer-Burggraf machine (Machine No. 2), the rate of application of stress, within the range studied, has no effect on test results. This range will cover any accidental variation in rate if the rate is initially adjusted so as to be near the middle of the range.

Abrams' method of third-point loading gives the lowest results of all of the different methods of flexural testing used in this investigation.

The various methods of flexural testing, each of which is used by one or more testing agencies, give widely varying results, indicating the need for a standard method of making this test.

DISCUSSION

Mr. Myers. MR. BERT MYERS[1] (*presented in written form*).—The paper by Messrs. Willis and Wray states quite clearly the situation which now exists with reference to the attempts being made to determine the flexural strength of plain concrete. It points out the wide variations in the design of the various machines being used for this purpose and the wide variation in results that may be expected.

In the design of a machine for the determination of the flexural strength of any material there are certain principles which should be considered. In the attempt to secure a testing machine for field use which would be portable, cheap and simple in design and operation, some of these fundamental principles of design have sometimes been overlooked. Some of these principles which apply to the design of a machine for the determination of the flexural strength of concrete specimen are as follows:

1. There should be no forces applied to the specimen except at right angles to the axis of the specimen.
2. All the forces applied to the specimen should be parallel to each other.
3. All the forces applied to the specimen should be measured by the testing machine.
4. The load should be applied at a uniform rate without impact.

A machine which seems to conform to these requirements has been designed and built by the Department of Materials and Tests of the Iowa State Highway Commission. In this machine, illustrated in the accompanying Fig. 1, the specimen is tested as a simple beam with the load applied at the center. The end bearings are supported on stirrups which are hung from links so that they are free to move in any direction. One of the end bearings is supported by a system of weighing levers. The load is applied by means of a geared jack actuated by a crank.

The measured reaction is balanced by a pail suspended from the end of the weighing lever. Water flows into this pail from a tank at the top of the machine. The water flows from the tank through a flexible rubber tube which passes over a support and under the lever from which the pail is supported. The load is applied at such a rate that the weighing beam is kept balanced. When the specimen breaks,

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[1] Engineer of Materials and Tests, Iowa State Highway Commission, Ames, Iowa.

Mr. Myers.

the weighing lever falls and the supply of water is automatically cut off. The pail is then transferred to the rear end of the weighing beam and is weighed by means of this beam.

Since this machine has not been in use for any great length of time, there is not available any great amount of data as to comparative results with other types of machines. The accompanying Table I gives comparative data as to the flexural strength of a single series of specimens consisting of twenty specimens tested on each of four types of machines.

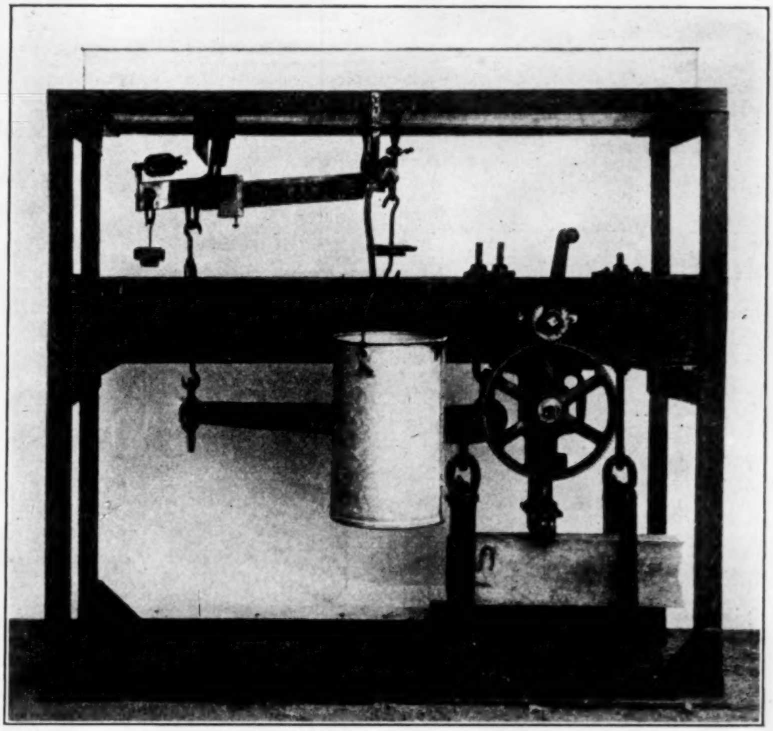

FIG. 1.—Flexure Testing Machine, Iowa State Highway Commission.

These data show that for a machine with knife edge bearings, similar to machine No. 1 in the paper, the maximum modulus of rupture was 620 lb. per sq. in., the minimum 418 lb. per sq. in. and the average 519. The average percentage variation from the average was 8.4 per cent.

For a machine similar in principle to machine No. 1 with roller bearings, the maximum modulus of rupture was 656, the minimum 462

Mr. Myers.

and the average 584 lb. per sq. in. The average percentage variation from the average was 6.6. For a machine similar in principle to the third point loading machine with a total span of 24 in., the maximum modulus of rupture was 669 lb. per sq. in., the minimum 454 and the average 558. The average percentage variation from the average was 9.3. For the machine with the beam supported from stirrups, the maximum modulus of rupture was 667, the minimum 527 and the average 588. The average percentage variation from the average was 5.1.

TABLE I.—RESULTS OF FLEXURAL TESTS OF CONCRETE BEAMS IN DIFFERENT TYPES OF TESTING MACHINES.

Size of beam 6 by 6 by 30 in.
Proportion 1:1.93:2.91 by weight.
Moist room storage.

Age seven days.
Water-cement ratio, 0.613.
Specimens tested wet.

Reaction Loading, 15-in. span						Third-Point Loading, 24-in. span			Mid-Span Loading 15-in. span		
Fixed Lever			Free Lever								
Beam	Modulus of Rupture, lb. per sq. in.	Variation from Average, per cent	Beam	Modulus of Rupture, lb. per sq. in.	Variation from Average, per cent	Beam	Modulus of Rupture, lb. per sq. in.	Variation from Average, per cent	Beam	Modulus of Rupture, lb. per sq. in.	Variation from Average, per cent
No. 15-1	510	1.7	No. 14-1	608	4.0	No. 11	615	10.2	No. 13-1	667	13.4
No. 15-2	621	21.0	No. 14-2	656	12.4	No. 12	604	8.2	No. 13-2	590	0.3
No. 20-1	418	19.5	No 19-1	605	3.6	No. 16	454	19.5	No. 18-1	527	10.4
No. 20-2	489	5.8	No. 19-2	610	4.5	No. 17	614	10.1	No. 18-2	612	4.1
No. 25-1	585	12.6	No. 24-1	518	5.8	No. 21	669	19.9	No. 23-1	589	0.2
No. 25-2	512	1.4	No. 24-2	642	10.0	No. 22	471	15.6	No. 23-2	621	5.6
No. 30-1	522	0.5	No. 29-1	613	5.0	No. 26	660	18.2	No. 28-1	...	...
No. 30-2	553	6.4	No. 29-2	579	1.0	No. 27	662	18.6	No. 28-2	607	3.2
No. 35-1	690	33.0	No. 34-1	612	4.8	No. 31	567	1.7	No. 33-1	625	6.3
No. 35-2	514	0.9	No. 34-2	577	1.2	No. 32	523	6.2	No. 33-2	559	4.9
No. 40-1	467	10.0	No. 39-1	462	21.0	No. 36	564	1.1	No. 38-1	582	1.0
No. 40-2	546	5.1	No. 39-2	612	4.8	No. 37	582	4.2	No. 38-2	562	4.4
No. 45-1	499	3.7	No. 44-1	523	10.5	No. 41	528	5.3	No .43-1	557	5.3
No. 45-2	468	9.8	No. 44-2	594	1.8	No. 42	517	7.3	No. 43-2	585	0.5
No. 50-1	509	2.0	No. 49-1	596	2.2	No. 46	563	0.9	No. 48-1	590	0.3
No. 50-2	511	1.6	No 49-2	...	....	No. 47	544	2.5	No. 48-2	646	9.9
No. 55-1	493	5.0	No. 54-1	488	16.5	No. 51	492	11.8	No. 53-1	597	1.5
No. 55-2	...	....	No. 54-2	513	1.9	No. 52	551	1.2	No. 53-2	528	10.2
No. 60-1	464	10.5	No. 59-1	591	5.2	No. 56	491	12.0	No 58-1	573	2.6
No. 60-2	472	9.1	No. 59-1	635	8.7	No. 57	496	11.1	No. 58-2	564	4.1
Maximum	628	21.0		656	12.4		669	19.9		667	13.4
Minimum	418	19.5		462	21.0		454	19.5		527	10.4
Average	519	8.4		584	6.6		558	9.3		588	5.1

From these rather meager data it would appear that a machine in which the principles mentioned are observed is capable of giving more uniform results than a machine which does not conform closely to these principles.

Mr. Lang.

MR. F. C. LANG[1] (*presented in written form*).—The information presented in the paper by Messrs. Willis and Wray is very timely. In 1927 I made inquiries of a number of State Highway Departments as to the type of equipment used in making the field transverse test

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[1] Engineer of Tests and Inspection, Minnesota Highway Department, St. Paul, Minn.

Mr. Lang. on concrete and their methods. I found that practically every state had a machine of its own and a different method. In general, however, the machines conformed to the types given as machine No. 1 and machine No. 2 in this paper.

Machine No. 1 should be used only with roller bearings. We once made the mistake of trying to have this machine with rigid bearings as used by the authors in methods I, II, and III, and we abandoned it for the reasons given by the authors. I found from my inquiries that some states were still using rigid bearings and there were some other mechanical defects which just as seriously impaired results.

At present the modulus of rupture as obtained in one state cannot be compared with that obtained in another, and the relationship between modulus of rupture obtained and that used in design is doubtful. We made up some beams and tested with 15-in. span center load on the same machine with rollers and with rigid bearings. With the rollers we obtained a modulus of rupture of 626 and with rigid bearings 984 lb. per sq. in. This is about as shown by the authors for method I and method IV with a 12-in. span.

I am of the opinion that rigid bearings should not be used and consequently I think they should not be considered when discussing the effect of length of span. Omitting methods I, II, and III, there is little in this paper to show the effect of length of span. Such information as there is really shows that with the center load there is not much difference. I think more information is needed on the effect of length of span, size of specimen, and rate of application of the load.

In using machine No. 1, I think there should be some pad between the concrete beam and metal plate; also that there should be a ball so as to assure uniform application of the load across the beam. With machine No. 2 I think plaster of Paris should be used at bearings.

Mr. Goldbeck. Mr. A. T. Goldbeck[1] (*by letter*).—The results of the tests quoted in the paper by Messrs. Willis and Wray are of very great interest at the present time because of the importance of the flexural strength of concrete for concrete highway construction. The studies made by the authors, showing the variation in results when different types of testing machines are used, are very timely and should serve as a warning to those who are building or purchasing flexure testing machines. It seems to be evident that the variations in results are due largely to the variations in testing machine design, and it should be of some value to analyze some of the designs now being used, with the idea of detecting possible errors. Several of the types of machines now in

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[1] Director, Bureau of Engineering, National Crushed Stone Association, Washington, D. C.

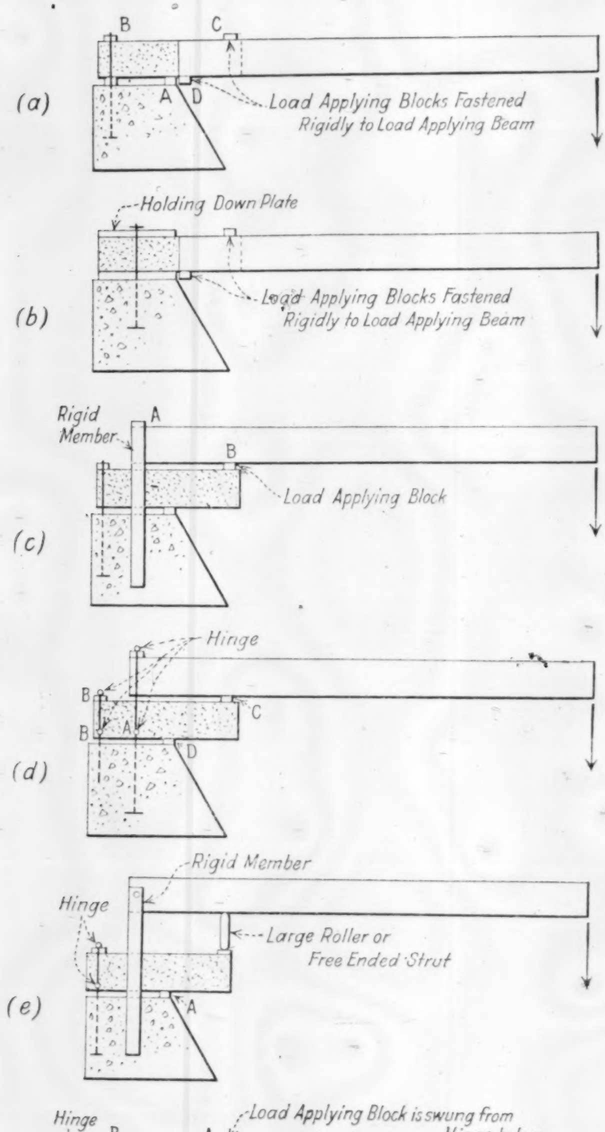
(a)
B
C
A
D
Load Applying Blocks Fastened Rigidly to Load Applying Beam
(b)
Holding Down Plate
Load Applying Blocks Fastened Rigidly to Load Applying Beam
(c)
Rigid Member
A
B
Load Applying Block
(d)
Hinge
B
B
A
C
D
(e)
Rigid Member
Hinge
Large Roller or Free Ended Strut
A
(f)
Hinge
B
A
Load Applying Block is swung from Hinge below
Hinge

FIG. 2.—Diagrams of Six Different Types of Machines Used in Cantilever Tests of Concrete Beams.

Mr. Goldbeck.

use are shown in diagrammatic form in the accompanying Figs. 2 (*a*), (*b*) and (*c*).

In Fig. 2 (*a*) the load is applied to the beam by means of an auxiliary beam which is hung over it at one end. Often the load-applying beam is made of wood. Let it be assumed that the concrete specimen is stressed up to 600 lb. per sq. in. at the "dangerous section," and that the length of the overhanging portion of this beam is 10 in. The total elongation of the upper fiber of the concrete beam then equals $\frac{300 \times 10}{3{,}000{,}000} = 0.001$ in. Let it be assumed that the wooden load-applying beam is stressed to 1000 lb. per sq. in. when the concrete specimen is stressed to 600 lb. per sq. in. The total stretch in the upper fibers of the wood load-applying beam is then $\frac{500 \times 10}{1{,}500{,}000} = 0.0033$ in. Suppose load-applying blocks *C* and *D* are fastened rigidly to the wooden load-applying beam. Of necessity, there will then be motion of these load-applying blocks relative to the concrete specimen. If lower load-applying block *D* does not slip with regard to the specimen, upper load-applying block *C* must slip outward, and a horizontal force as well as a vertical force will be produced on the specimen at this point. It is quite evident that the magnitude of this horizontal force will vary, depending upon a number of factors, such as the magnitude of the load applied, the kind of material used in the load-applying beam, the design of the load-applying beam, etc. Although it is clear that the calculated stress at the "dangerous section" depends upon the value of the bending moment which in turn is definite, none the less it is also true that horizontal local stresses are induced of unknown magnitude and of unknown effect and, because of the uncertainty introduced by the horizontal components of the forces at *C* and *D*, it would seem well to avoid the use of machines of this type.

A variation of the machine shown in Fig. 2 (*a*) is that in Fig. 2 (*b*). In this case, the specimen is clamped down to its support by means of a plate, and it rests upon the support over a considerable length. With this type of support, there is the possibility of a very considerable variation in the distribution of the vertical reaction pressures. For illustration, if the holding-down bolts are made very tight, there will result rather high vertical pressures throughout the entire length of support. On the other hand, if these bolts are made very loose, when load is applied, there will be no vertical force at one end, and a maximum force at the other end. Although this has no effect whatever on the bending moment and on the stress calculated from that bending moment, there is some uncertainty introduced

Mr. Goldbeck.

because of the variation in local stresses at the "dangerous section." Such a machine would be difficult to standardize, and therefore should be avoided in favor of a type of machine which lends itself to positive standardization.

In Fig. 2 (*c*) is shown still another type of machine which has been largely used. In this case, the load-applying beam is fastened to a rigid member which, in turn, is anchored to the support for the test specimen. The lower fibers of the load-applying beam are in compression, while the upper fibers of the concrete test specimen are in tension. Therefore, there will be relative motion of load-applying block *B* with regard to the concrete test specimen and consequently a horizontal force is produced on the test specimen at the point *B*. This force will tend to decrease the tension in the specimen, produced by bending. It would be expected that a machine of this type would give high results for modulus of rupture, and the test results show that this is actually the case.

In laboratory testing of beams, a simple beam specimen, instead of a cantilever beam, is frequently used. Sometimes the load is applied as a single, concentrated load, at others two-point loading is used, as illustrated in Fig. 3 of the paper by Messrs. Willis and Wray. In the simple beam form of test specimen there is possibility of error due to the introduction of horizontal components at the points of reaction and at the points of load application, just as in the case of the cantilever beam specimen. From a theoretical standpoint, the set-up shown in Fig. 3 is subject to criticism as follows: In the first place, symmetrical elastic action of the test specimen is not possible because of the different conditions existing on each side of the center line of the specimen. Thus a roller support is provided at the right and a spherical bearing block support at the left of the specimen. At one of the points of loading, a roller is provided, while at the other point a spherical bearing block is used. A spherical bearing block does not permit free horizontal motion to take place, and thus there is the possibility of the introduction of a horizontal component at the left reaction and at the right point of load application. The magnitude of these horizontal forces is uncertain, and it would seem that on this account steps should be taken to so design a set-up that nothing but vertical forces will be possible. Cantilever testing machines, which apply nothing but vertical forces to the test specimen are illustrated diagrammatically in the accompanying Figs. 2 (*d*), (*e*) and (*f*).

In Fig. 5 (*d*) it will be noted that the load is applied at the end of the load-applying beam as usual and the other end is held down by means of hangers with hinges as shown. The test specimen is likewise

Mr. Goldbeck.

held down at one end by means of hinged hangers. The force holding down the end of the load-applying beam must act in a vertical direction *AA* through the two hinges. Since the load is vertical, the force applied on the specimen at load-applying block *C* must also be vertical. The reaction at the end of the test specimen must also act in a vertical direction *BB*. As the force *C* is vertical, and the reaction *BB* is vertical also, the reaction at point *D* must also be vertical. Thus, nothing but vertical external forces act upon the test specimen and no uncertainty is introduced through the production of horizontal components of unknown amounts.

In Fig. 2 (*e*) a large roller or a free-ended strut is provided at the end of the test specimen. The load acting at the end of the test specimen, under this arrangement, will be in a vertical direction. The reaction at the other end of the test specimen will be vertical since it must act through the hinges provided in the holding-down mechanism. Consequently, the force acting in the load-supporting block *A* must likewise be vertical. Here, again, only vertical external forces act upon the test specimen.

In Fig. 2 (*f*) there is a modification of the arrangement shown in Fig. 2 (*a*), which is now so widely used. The load-applying block *A* is suspended from a hinge or by means of flexible hangers from the under side of the load-applying beam. The test specimen is held down, not by rigid bolts, but by a flexible hanger or by a tension member provided with a hinge. With this arrangement, vertical external forces only can act upon the test specimen.

In conclusion, it would seem desirable that:

1. Beam testing machines be so designed that any possibility of horizontal components of loads or reactions be eliminated, to the end that only vertical external forces may act upon the test specimen.

2. That the set-up of the specimen be arranged in such a manner that it may be duplicated elsewhere without difficulty.

Reference should be made to Appendix I to the report of Committee D-4 on Road and Paving Materials[1] for recommendations governing the transverse testing of concrete. It should go without saying that precautions should be taken:

(*a*) To avoid eccentricity of loading;

(*b*) To provide uniformity of rate of application of loading; and

(*c*) To eliminate the possibility of error due to friction in the weighing of the loads.

Too often it is assumed that field testing is necessarily of a rough character, and not subject to the ordinary refinements practiced in

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 527 (1928).

Mr. Goldbeck.

the laboratory. This assumption is entirely erroneous. It is entirely possible to introduce refinement in field testing just as in the laboratory, keeping in mind, however, that testing is a very practical matter and that refinement may be carried too far. When it is just as easy, or practically as easy, to design and build testing machines which eliminate uncertainty, certainly this should be done rather than to use ill-conceived and crude devices which may give results whose value is marred by uncertainty.

Messrs. Willis and Wray.

MESSRS. T. F. WILLIS[1] AND F. N. WRAY[1] (*authors' closure by letter*).—The discussions of our paper by Messrs. Myers, Lang and Goldbeck have added considerable force to the principal object, namely, to bring into prominence the discrepancies introduced in flexure testing by certain features of the apparatus.

Referring to Mr. Myers' discussion, the relations between the averages for the different methods of loading, as given in his table, differ considerably from those which we obtained. He obtains a higher average with a free lever arm than with a fixed lever arm, and the latter is lower than the third-point method or the freely swung beam loaded at the mid-point. We are unable to account for these differences and they do not seem to be in accord with theory.

In the last paragraph of Mr. Lang's discussion, speaking of our machine No. 1, he says that a ball should be used in the bearing block to insure uniform application of load across the specimen. We wish to state that both of the bearing plates on top of the concrete beam, as shown in Fig. 1 of our paper, are equipped so as to give the equivalent of ball action. The bottom bearing, which is under the point of maximum moment in the specimen, acts as a knife edge and stands rigidly horizontal. Full line contact between the concrete and the knife edge is desirable and the best way to obtain that condition is to provide a straight, level knife edge and then mold the beams so as to have a truly plane surface on the bottom. It is our opinion that the use of pads between the bearing plates and the concrete, particularly on the compression side of beams, is not vital to the test.

Concerning Mr. Goldbeck's discussion, we are in accord with his comments on various types of machines. Each of the machines shown in Figs. 2 (*c*), (*d*) and (*e*), is of the pivoted lever arm type. As shown in the illustrations, the pivot point does not stand directly above the plane of maximum moment in the concrete beam. We feel that there is a distinct advantage in having the lever pivoted

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[1] Bureau of Materials, Missouri State Highway Commission, Jefferson City, Mo.

Messrs. Willis and Wray.

vertically above this plane, because the total moment is much more easily calculated. It obviates the necessity of considering difference or variation in the length of the beam between the lower knife edge and the forward top bearing plate, and the applied moment (not including the initial dead load moment) is always equal to the applied load times the length of the load applying lever.

EXPERIMENTAL TESTS OF CONCRETE-STEEL BOND

By L. N. Edwards[1] and H. L. Greenleaf[2]

Synopsis

The safety and dependability of reinforced-concrete structures are predicated alike upon the adequacy of their design and the efficiency of their construction. The experimental tests described in this paper were made with the primary object of securing information relating to the bond produced between concrete and steel reinforcement bars under conditions, as far as practicable, in conformity with the best field construction practice. The results obtained show:

1. A wide range of bond strength, probably indicating the effect of the "mother rock" origin of the sands used and of their granulometric composition.
2. The general effect of the use of an excessive water content in the concrete mix.
3. The desirability of a more complete investigation of the factors affecting concrete-steel bond and its reliability under conditions involving variations in the application of loads and impact.

Introduction

The experimental work described in this paper was suggested several years ago by the results of tests made by one of the authors[3]; however, it is only recently that an opportunity has been found to undertake even in a limited way the necessary tests to extend that earlier investigation. In the earlier paper it was rather clearly shown that the strength of the concrete-steel bond is affected by the quantity of water used in the concrete mix. For convenience of reference, Fig. 1 of this paper is reproduced therefrom with slight modification, the details of the test beam used being combined with the strength curve diagram.

In the tests herein described the method of placing the steel bars differed from that which has been commonly used by other investigators, in that the bars were inserted in a horizontal position in the forms and the concrete compacted around them instead of the bars being placed in a vertical position. The object in placing the bars in a horizontal position was to secure information conforming to actual field construction conditions wherein the reinforcing steel of the slabs and beams of bridges, roadway slabs of highways, girders and floor slabs of buildings and other similar constructions is placed in horizontal and inclined positions rather than in a vertical position as it ordinarily exists in columns.

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[1] **Bridge Engineer**, State Highway Commission, Augusta, Me.

[2] **Bridge Inspector**, Maine State Highway Commission, Monmouth, Me.

[3] L. N. Edwards, "Effects of Grading of Sands and Consistency of Mix upon the Strength of Plain and Reinforced Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XVII, Part II, p. 301(1917).

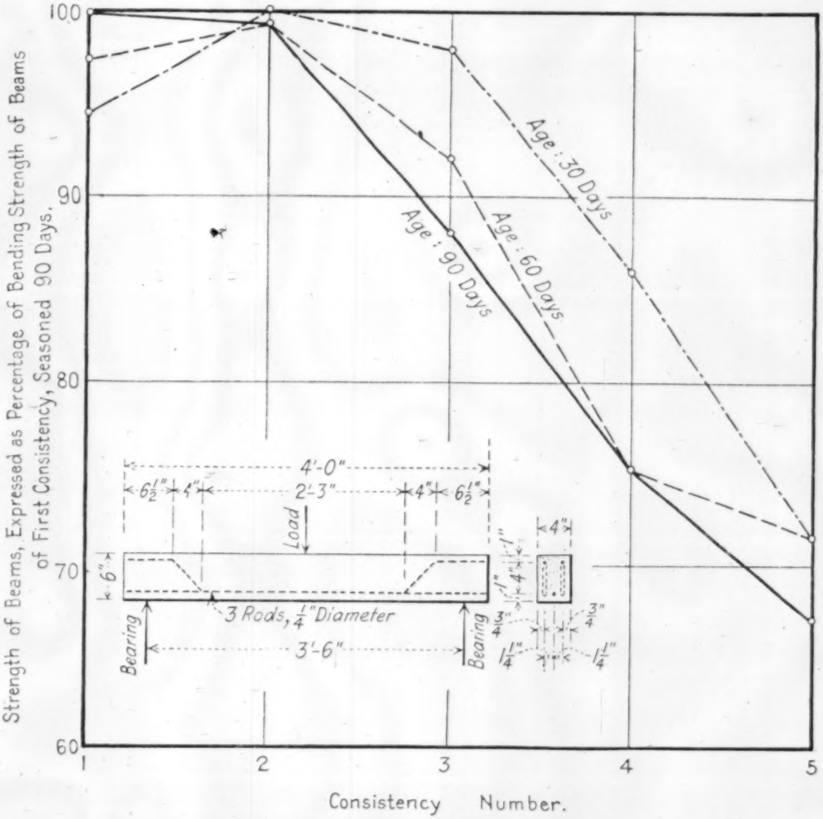


FIG. 1.—Details of Reinforced-Concrete Beams and Diagram Showing Strengths of Beams of Varying Consistencies and Ages.

Consistency 1.—A concrete having normal mobility or consistency.

Consistencies 2, 3, 4 and 5.—Concretes in which the water content was increased 10, 20, 35 and 50 per cent, respectively, over that used in the concrete of normal mobility.

The three main factors involved in these tests are as follows:

1. The effect of varying the quantity of cement in the concrete mix;
2. The effect of varying the quantity of water in the mix; and
3. The effect of using sands having different origins, both as regards their "mother rock" condition and the locations of pits.

Materials

The properties of the cement, sand and gravel were as follows:

Cement:

A standard brand of portland cement fulfilling all requirements of A.S.T.M. Specifications C 9 – 26 was used, the entire quantity being secured from a single carload lot.

Sands:

In the test involving varying quantities of mixing water, Cottle Pit sand was used. In the tests involving varying quantities of cement and different sands the other four sands described below were used:

Cottle Pit, Augusta, Me.—Composed of about 75-per-cent granitic material, the remainder being for the most part fine-grained, dark-colored, igneous and schistose rock material.

Decker Pit, Lagrange, Me.—Composed essentially of 90-per-cent fine-grained igneous rock, the remaining portion being granitic. This sand shows evidence of iron in its make-up.

Cole Pit, Fairfield, Me.—Composed essentially of 90-per-cent fine-grained igneous and schistose rock material, the remainder being granitic, there being evidence of iron.

Graham Pit, Bangor, Me.—Composed of nearly 100-per-cent fine-grained, dark-colored igneous rock.

Jack Pit, Lisbon, Me.—Composed of nearly 100-per-cent granitic material, very angular.

The following tabulation shows the granulometric composition of the sands as determined from a single laboratory sample of each:

	Sieve Analysis, per cent				
Sieve (W. S. Tyler Series)	Cottle Pit Sand	Decker Pit Sand	Cole Pit Sand	Graham Pit Sand	Jack Pit Sand
Passing No. 4, Retained on No. 8	10.7	11.0	6.0	33.0	16.0
Passing No. 8, Retained on No. 14	31.3	35.0	24.0	30.0	23.0
Passing No. 14, Retained on No. 28	37.6	38.0	37.0	19.0	27.0
Passing No. 28, Retained on No. 48	15.2	12.0	20.0	11.0	23.0
Passing No. 48, Retained on No. 100	2.9	2.0	5.0	3.0	7.0
Passing No. 100	2.3	2.0	8.0	4.0	4.0
	100.0	100.0	100.0	100.0	100.0

It is important to bear in mind that the foregoing tabulation is indicative only of the general composition of the sands, since tests have clearly shown that consecutive samples taken from the same laboratory sample may vary rather widely and, furthermore, that the range of variation is greater when specimens are taken from larger volumes.

These sands gave the following standard test strength results, in which the mortar mixes were 1:3 for tension tests and 1:2 for compression, the comparisons being made with corresponding standard Ottawa sand mixes. All relations are expressed in percentages of the latter.

Pit	Tensile Strength, per cent of standard 7-day	Tensile Strength, per cent of standard 28-day	Compressive Strength, per cent of standard 7-day	Compressive Strength, per cent of standard 28-day
Cottle	106.7	91.6	114.0	90.5
Decker	108.2	113.1	109.6	104.6
Cole	116.0	112.9	99.0	103.6
Graham	140.3	123.0	121.2	115.7
Jack	139.0	109.0	121.0	116.5

These data are given for those who perhaps place greater reliance upon their significance than the authors.

Gravel:

The stone aggregate was secured from Graham Pit, Bangor, Me. Its particles for the most part were made up of well-rounded pebbles, the greater portion of which was derived from granite and fine-grained igneous "mother rock" origins. This material was separated into sizes retained on a ¼-in. sieve and passing sieves having ½-in., 1-in. and 1¼-in. openings, and was recombined in equal parts by weight of each size.

PREPARATION OF TEST SPECIMENS

In the preparation of test specimens, including the operations incident to making the concrete, the operators were men thoroughly competent to judge as to the practical conditions involved, since they had been employed as inspectors throughout four consecutive construction seasons upon bridge construction work wherein the methods used were unquestionably high class in all respects.

Throughout the entire work of preparing test specimens, care was taken to conform as nearly as practicable to the conditions encountered in the actual field operations incident to building bridges and other ordinary reinforced-concrete structures.

Design and Proportioning of Concrete:

In the proportioning of the batches of concrete the quantities of materials were determined by weight. The following weights were used: cement, 376 lb. per bbl.; sand, 100 lb. per cu. ft.; and stone, 100 lb. per cu. ft. The sand and stone aggregates were surface dried.

In the concrete mixes using the four different sands the cement content was varied in the relations 1.35, 1.45, 1.55 and 1.65 bbl. per cu. yd. The mobility of the mixes was kept constant.

The following tabulation shows the quantities of materials used in the batches of concrete with the sand secured from Decker Pit:

	CEMENT CONTENT, BBL. PER CU. YD.			
	1.35	1.45	1.55	1.65
Cement, lb	54	58.5	60	63
Sand (Decker Pit), lb	120	125	118	117
Stone, lb	240	250	236	234
Water, lb	32	32	35.5	33.5

With the sands secured from the other three pits, the quantity of water used was varied in order to secure uniform mobility; in other words, the quantity of water in the concrete was found to be dependent upon the sand used. The following tabulation shows the quantities of water used in each mix; the quantities of cement, sand and stone aggregates being the same as described for the mixes in which Decker Pit sand was used:

	WATER USED IN MIXES, LB.			
CEMENT CONTENT, BBL. PER CU. YD.	DECKER PIT SAND	COLE PIT SAND	GRAHAM PIT SAND	JACK PIT SAND
1.35	32.0	33.0	36.0	32.5
1.45	32.0	34.0	36.0	33.0
1.55	35.5	36.0	35.0	34.0
1.65	33.5	36.0	35.0	34.5

The above tabulation shows that for each sand the quantity of water required to produce the desired mobility in the mixes containing 1.55 bbl. of cement per cubic yard was relatively greater than was required for the 1.65-bbl.-per-cu.-yd. mixes. This condition is not readily explained, more especially when consideration is given to the fact that retests, independently made, checked very closely the quantities of water used in the original tests.

In the tests made to determine the effect of varying the quantity of water in the mix, the following quantities of materials were used in the normal or initial concrete mix: Cement, 60 lb.; sand (Cottle Pit), 120 lb.; stone, 240 lb.; water, 30 lb. The quantity of water in relation to that contained in the normal workable mix was increased 10, 20, 35 and 50 per cent in the wetter or succeeding mixes.

Mixing of Concrete:

The concrete was mixed in a Koehring electrically driven one-half bag batch mixer. In preparation for mixing the concrete the interior surfaces of the mixer drum were thoroughly coated with mortar by using a "priming" charge of sand, cement and water of the same composition as the mortar to be used in the specimens. All excess mortar was discharged from the mixer.

In order to determine the quantities of water required in the various mixes to produce a mortar component having a mobility that may be described as a saturated, sticky, semi-plastic material showing no free water, trial mixes were made. This condition was determined by visual examination. When measured by the slump and the flow table methods the slump was about 1½ in. and the flow about 1.29. The foregoing conditions apply to all test batches except those in which excess water was used.

After the interior of the mixer drum had been coated, the quantity of water required for a batch (as determined by the test batches) was placed in the mixer, the necessary quantities of cement and sand and stone aggregates were introduced and the entire charge was mixed 1½ minutes after all materials were placed in the drum. In order to produce ten concrete-bond specimens and five concrete test cylinders 6 in. in diameter by 12 in. in length, it was necessary to make two batches of the mixed concrete.

Forms, Placing and Storage:

The concrete-steel bond specimens were cast in wooden forms measuring 8 by 8 by 12 in. in inside dimensions. The steel rods 1 in. in diameter by 30 in. in length were of structural grade, plain, newly rolled from billet stock and free from rust. The bars were inserted horizontally through holes in the ends of the forms and so located that the rod projected 1 in. outside the concrete blocks at one end to permit the attachment of the measuring device. The 1-in. diameter rod was chosen in order that the ultimate stresses produced in the tests might be well within the elastic limit of the metal.

The placing of the concrete in the forms was carefully done by the operators, all compacting being secured by the use of steel trowels; and when the forms had been completely filled the top surface of the concrete was screeded flush with the top edges of the forms and was then troweled smooth.

The test cylinders were made by compacting the concrete as the filling of the cylinder forms progressed, a rod, ⅜ in. in diameter, flattened at its end to form a blade-like tool, being used for the purpose.

The test specimens, except those made with varying water content, were removed from the forms at the age of 24 hours and were immediately stored in a moist room maintained at a temperature ranging from 66 to 70° F., where they remained until removed for testing. The test specimens made with varying quantities of water were in general removed from the forms at the age of 24 hours, but those containing 35 and 50 per cent excess water were found to lack the strength necessary to permit satisfactory removal of forms until a period of practically 48 hours had elapsed. These specimens were stored under burlap, kept constantly wet for a period of 10 days and were then permitted to cure in the open air of the laboratory until tested.

For the tests herein described there were made 210 concrete-steel bond specimens and 105 concrete cylinders.

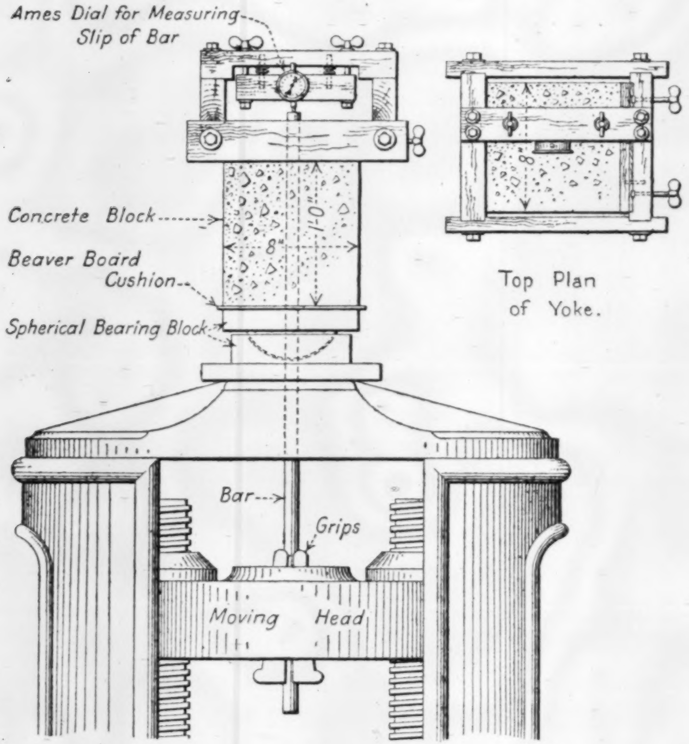

Ames Dial for Measuring Slip of Bar
Concrete Block
Beaver Board Cushion
Spherical Bearing Block
1'-0"
8"
Top Plan of Yoke.
Bar
Grips
Moving Head

FIG. 2.—Details of Device Used in Testing Concrete-Steel Bond Specimens.

APPARATUS AND METHODS OF TEST

A Riehle testing machine of 60,000-lb. capacity was used in the bond tests after adapting it to this special purpose by placing a spherical bearing block upon the fixed head. The construction of this block permits the rod of the specimen to extend downward through it and to engage the grips in the movable head of the machine so that by moving the head downward the rod is pulled out of the concrete block. Figure 2 illustrates this device, together with the method of applying a yoke upon the uppermost end of the concrete specimen by which an Ames dial reading to 0.001 in. was held in place to record the amount of the slip or movement of the bar resulting from the loads applied at the movable head. Readings of the slip of the bar were taken at intervals of 0.005 in. up to 0.025 in. and at intervals thereafter of 0.05 in. up to 0.30 in.; correspondingly the loads applied upon the movable head of the machine were recorded.

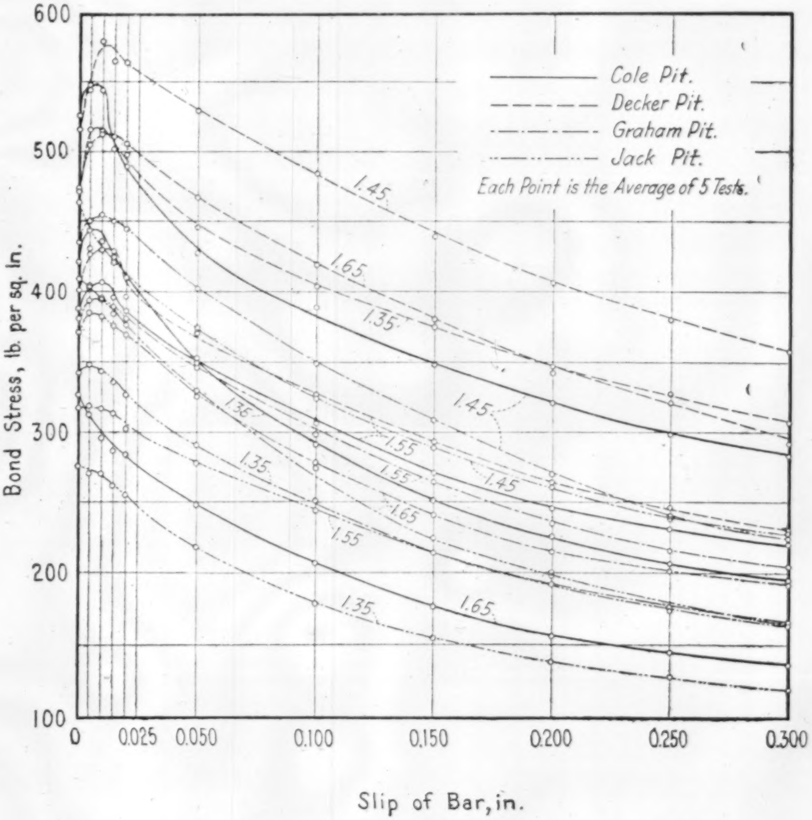


FIG. 3.—Load-Slip Curves Showing Relation of Concrete-Steel Bond Stress to Cement Content of Mix. Four Different Sands. Age 1 year.

Concrete cylinders were broken in a Riehle testing machine of 150,000-lb. capacity.

TEST DATA AND DISCUSSION

With the object of conserving space and of visualizing the results of the tests, the bond-slip data are presented in graphical form. With the exception of Fig. 4, the circles shown upon all diagrams indicate the bond strengths secured as an average of five test specimens.

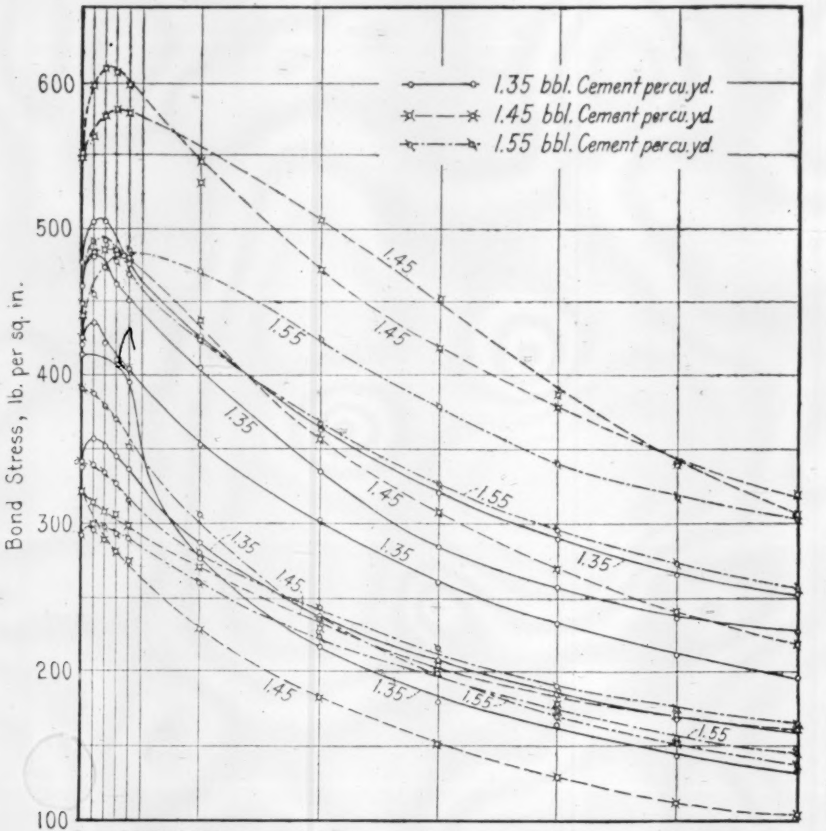


FIG. 4.—Load-Slip Curves of Individual Test Specimens Using Graham Pit Sand. Five Specimens for Each of Three Different Cement Contents of Mix. Age 1 year.

Figure 3 shows the bond-slip relations secured from one-year tests in which the cement content of the mix was varied from 1.35 to 1.65 bbl. per cu. yd. and in which four different sands were used.

Figure 4 shows the bond-slip relations secured from the individual test specimens of the Graham Pit sand at the age of one year. The data of the 1.65 cement-content specimens are omitted in order to render the diagram more legible.

Figure 5 shows the bond-slip relations secured from 28-day and 1-year tests in which the quantity of water in the mix was varied from a normal workable mix to a mix containing a 50-per-cent excess quantity of water.

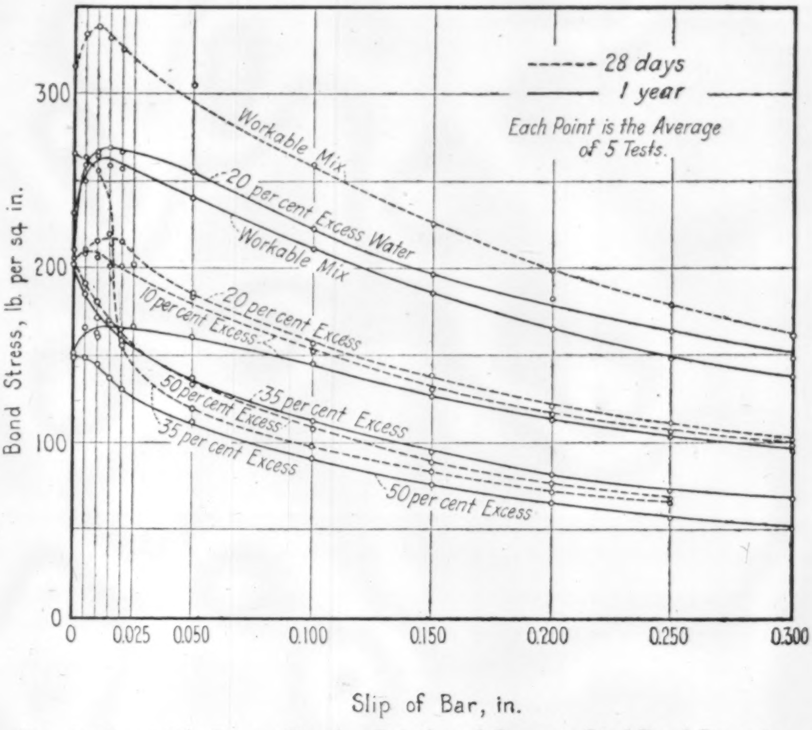


FIG. 5.—Load-Slip Curves Showing Relation of Concrete-Steel Bond Stress to Water Content of Mix, Using Cottle Pit Sand.

While the results obtained by these investigations may appeal to some as erratic and unexplainable and may, therefore, be regarded as indicative of the existence of carelessness and inattention to detail on the part of the operators, we know that on the contrary the details of the operations were given full and due attention and that the work as a whole was executed in strict accordance with first-class field methods. We venture to believe that the results indicate clearly the wide variation that under corresponding conditions are actually being produced in field work and, furthermore, that the loose, slipshod and otherwise unsatisfactory conditions too frequently found in actual construction work must inevitably produce results more widely variable and undependable.

It is a well-recognized condition that machine-mixed concrete produces greater variability in specimens than does hand-mixed concrete. However, in these tests field construction methods are an important factor.

Ratio of Bond and Compressive Strengths:

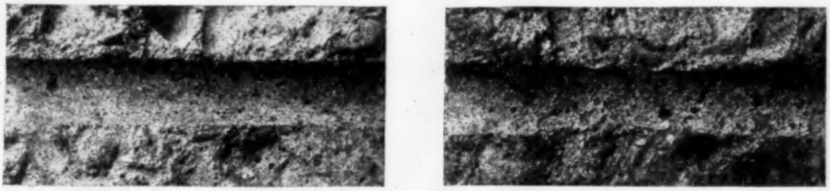
For the tests involving varying quantities of cement and four different sands, the concrete cylinders which were cast coincident with the bond specimens afford a means of computing a general, though rather widely varying relation between the cylinder strengths and the concrete-steel bond strengths at the age of 28 days. When expressed as percentages these relations varied from 11 to 19 per cent at 28 days, with an average of 15 per cent. In the following table, the compressive strengths of the cylinders are given, each strength representing an average of five test specimens.

COMPRESSION STRENGTHS OF CONCRETE CYLINDERS AT 28 DAYS, LB. PER SQ. IN.

CEMENT CONTENT, BBL. PER CU. YD.	DECKER PIT SAND	COLE PIT SAND	GRAHAM PIT SAND	JACK PIT SAND
1.35	1606	1463	1904	1986
1.45	2141	1925	2502	2294
1.55	1858	1904	2115	2199
1.65	2889	2316	2046	2694

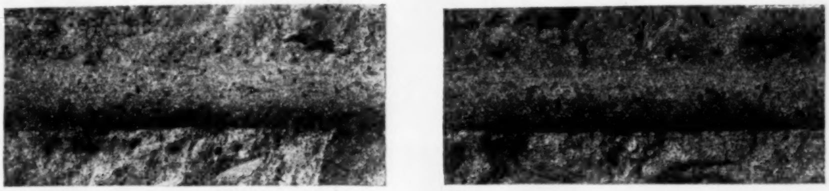
Character of Bond Failure:

When the specimens had been broken longitudinally through their center either in a plane vertical to the top (troweled) surface or horizontally at their mid-height, thus permitting the removal of the steel bar, the texture of the curved surface of the concrete surrounding the bar indicated the character of the bond. Upon the portion located above the bar the existing minute voids appear as mere pin holes, while upon the portion located beneath the bar there exists ample evidence that water accumulated in flattened globules in contact with the under portion of the bar, giving to this area a pitted, crater-covered appearance. In the specimens, the concrete of which was made with an excess water content, the water cups or cavities occupy a considerable percentage of the area beneath the bar, thereby diminishing to a very considerable extent the area of the bonded or contact surface. In this connection it is important to note that in all the specimens the polished surfaces produced by the slipping of the bars indicated clearly that the areas located beneath the extreme lower portion of the bars had little or no contact with the metal and that the movement of the bars left the water cups and cavities undisturbed.



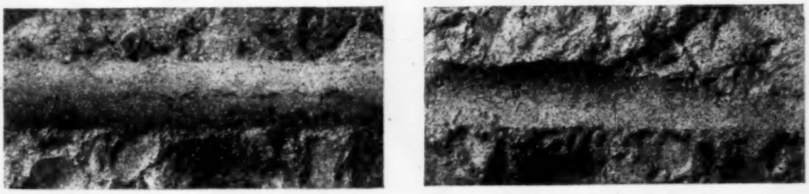
(*a*) Horizontal break. Shows average surface underneath bar. Workable mix. Polished areas indicate extent of bond.

(*b*) Horizontal break. Shows surface underneath bar in poorest specimen. Workable mix. Polished areas indicate extent of bond.



(*c*) Vertical break. Shows surface above and underneath bar. Workable mix. Absence of polished areas at extreme lower portion of curved surface indicates no contact between bar and concrete.

(*d*) Vertical break. Shows surface above and underneath bar. Excess water mix. Polished areas above center of bar indicate bond. Water cups and cavities below center of bar unaffected by its movement. No bond.



(*e*) Shows surfaces above and underneath bar in a single test specimen. Excess water mix. Polished areas indicate bond above bar. Water cups and cavities underneath bar unaffected by its movement.

FIG. 6.—Character of Mortar Contact or Bond.

Figure 6 shows in photographs the character of bond as existing in typical specimens.

The typical failure of the specimens involved (1) bond strength at point of initial slip and (2) a rise in this strength between the initial slip and a slip of 0.01 in., indicating, probably, the existence of a mechanical bond resulting from surface irregularities in the bars. This condition is clearly shown in Figs. 3 and 4.

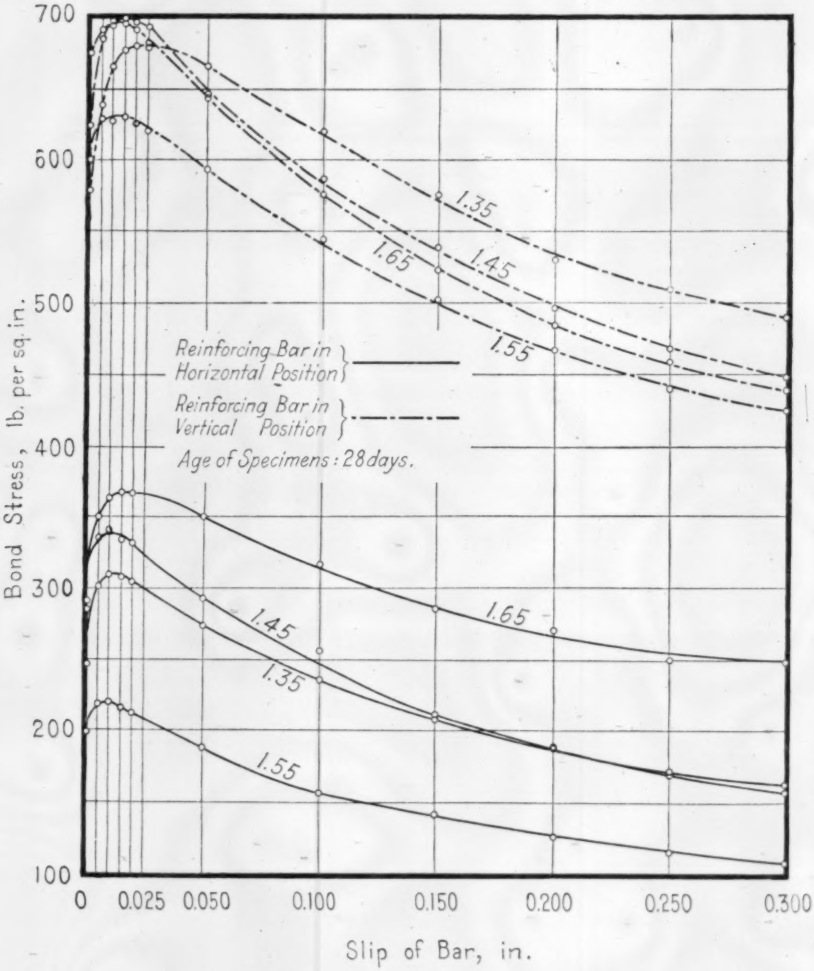


FIG. 7.—Load Slip Curves Showing Differences in Bond Stresses Resulting from Placing Bars in Horizontal and in Vertical Positions.

Four different cement contents of mix. Cottle Pit sand. Age 28 days.

Effect of Water Content:

The most important effect of increasing the quantity of water in the concrete mix is shown graphically in Fig. 5. The bond strength is considerably decreased by the increased water content of the mix and, furthermore, the specimens made from the wet mixes did not show a rise in strength after the initial slip had been produced. It is probable that void space beneath the bars resulting from entrapped water was sufficient to permit the bars to move freely after the initial bond failure. In the testing of some of the specimens the rapid movement of the bar following the initial slip rendered it impossible to secure the corresponding dial and load readings.

Incidental Observations:

In general, the tests at 28 days showed higher bond strengths than the 1-year tests. Possibly this condition was due to the complete drying out of the specimens and a resulting shrinkage partially breaking the bond contact.

The concrete mixes containing cement in the proportions 1.55 and 1.65 bbl. per cu. yd. gave, in general, lower bond strengths than did the mixes containing less cement. Among engineers and laboratory operators it is a well-known fact that so called rich or "fatty" concrete mixes are subject to greater volume change in the setting period and in the later drying out stage than are the leaner mixes. It seems likely, therefore, that the explanation of these low bond strengths may be found in the tendency of these mixes to shrink away from contact with the steel rods.

The concrete-steel bond strengths secured in this investigation differ widely from those reported by other investigators. This condition appears to be entirely due to the method used in preparing specimens. A small series of tests was made to permit a direct comparison of the two methods. Figure 7 shows the concrete-steel bond strengths at an age of 28 days secured from specimens in which the bars were inserted in a horizontal position in the forms and correspondingly the bond strengths resulting in specimens with bars placed in a vertical position.

CONCLUSIONS

As indicated in the introduction, this investigation is too limited in its scope to permit definite conclusions to be drawn therefrom. However, the following general conclusions appear justified from the data presented:

1. The "mother rock" origin and the granulometric composition of sands are factors affecting bond strength.
2. The cement content of the mix is a factor affecting the intensity of the concrete-steel bond. A high cement content may produce less bond strength than would a lesser one.
3. The variation of the water content of a concrete mix affects

the intensity of the concrete-steel bond. Assuming as a basis for comparison a normal, semi-plastic concrete mix, an increase in the water content will produce a change in the character of and a reduction in the bond strength.

4. The investigations made, although limited in scope and in the number of specimens tested, indicate the desirability of further investigations in which field construction methods and operations are given due consideration, together with a study of concrete-steel bond resistance to suddenly applied loads and impact.

DISCUSSION

Mr. E. F. Kelley[1] (*presented in written form*).—This report by Edwards and Greenleaf covers what is believed to be the most comprehensive investigation that has been made of "pull-out" concrete-steel bond tests in which the test specimens were cast with the bars in a horizontal position, simulating the more common construction condition in which the reinforcement is horizontal or inclined. The attention to detail, which characterized the investigation, is worthy of note. **Mr. Kelley.**

The examination of the broken specimens disclosed a condition that is not particularly surprising but the possible existence of which, in actual concrete construction is not usually appreciated or considered. The deficiency in bonded area on the under side of the test bars, which were plain round rods, leads one to speculate as to the condition which might have obtained had the bars been square in cross-section, thus producing a condition even more favorable for the formation of voids.

The observed lack of bonded area also leads one to inquire regarding the effect of this condition on the resulting bond strengths, as compared with those obtained from specimens cast in the usual manner; that is, with the rods placed vertically. The comparison is given in a striking manner in Fig. 7 which shows the average values for initial slip in specimens with the bars cast horizontally to be less than 50 per cent of those obtained with specimens in which the bars were cast vertically. In a similar investigation, involving a comparatively small number of specimens, Abrams[2] found the bond resistance of specimens molded in the horizontal position to be about 60 per cent of that for similar specimens with the bars in a vertical position.

As a result of an extensive series of tests on pull-out specimens cast with the bars in a vertical position, Abrams[3] reported initial slip at a bond stress of 10 to 15 per cent of the compressive strength of the concrete and a bond stress of 50 to 60 per cent of the ultimate bond at an end slip of 0.0005 in. The authors of the paper under discussion report a bond strength at 28 days varying from 11 to 19 per cent of the compressive strength, with an average of 15 per cent, and the graphs which are presented indicate that initial slip occurred at a stress of 85 to 100 per cent of the ultimate developed.

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[1] Chief, Division of Tests, U. S. Bureau of Public Roads, Washington, D. C.

[2] D. A. Abrams, "Tests of Bond Between Concrete and Steel," *Bulletin No. 71*, Engineering Experiment Station, University of Illinois.

[3] D. A. Abrams, "Studies of Bond Between Concrete and Steel," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 256 (1925).

Mr. Kelley. The initial bond strengths, expressed as percentages of compressive strength, therefore, are in fairly close agreement with those reported by Abrams. On the other hand the great difference in the relationship between initial and ultimate bond stress as shown in these tests and those of Abrams may be significant.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[1] recommends that the working stress for bond between concrete and plain reinforcement bars be taken as 4 per cent of the 28-day compressive strength of the concrete. For this reason the bond strengths reported may not be disturbing to the designer of concrete structures. However, the evident lack of bonded area on the under side of the bars can scarcely be ignored by those who are interested in good construction since such a condition is undoubtedly a source of weakness in the structure. Moreover, in this case, the construction of the specimens was in the hands of experienced men and an effort was made to simulate first-class construction methods. What may we expect when the concrete is carelessly placed around the reinforcement, as is too frequently the case under actual job conditions?

The writer can concur with the authors in the conclusion that further investigations in this field are desirable. Such investigations should cover not only present construction practices but also possible means for their improvement. The possibilities of improved reinforced concrete construction through the vibration of the forms and reinforcement with electric or pneumatic hammers is a promising field of investigation.

Mr. Slater. MR. W. A. SLATER[2].—It is probably true that there is no fundamental subject in the design of reinforced concrete on which information is so needed as on bond. The subject of reinforced concrete has been given little consideration in the past few years, in view of the importance which the subject of design of concrete itself has taken. I think Mr. Edwards does a service in bringing to our attention the need for more data of this kind. It is not the first time, however, that the fact of the separation between the concrete and the under side of the bar has been exhibited. Tests were made at the University of Illinois about 1912[3] on some beams in which the bars were situated near the top of the beam, with about fifteen inches of concrete under the bars. The concrete settled away showing a cavity under the bar in which there was no contact of the concrete with the bar at all, and the beams failed by bond.

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[1] Standard Specifications for Concrete and Reinforced Concrete, *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, p. 312 (1924).

[2] Research Professor of Engineering Materials and Director, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

[3] Richart and Larson, "An Investigation of Web Stresses in Reinforced Concrete Beams," *Bulletin No. 175*, Engineering Experiment Station, University of Illinois, p. 19 (1928).

Mr. Slater. I do not think it has been thoroughly established as to what form of bond specimen is most satisfactory. In the pull-out specimen, whether cast with the bar vertical or with the bar horizontal, it is not possible to determine except arbitrarily at what load or at what amount of slip a comparison between the different specimens should be made. If you take the first slip as a criterion for comparison between bars of different types, then there would be no difference in bond value between a smooth bar and a deformed bar, because the tests on record indicate that the two begin to slip under about the same bond stress, whereas if the maximum load carried be used as the criterion, too much credit will be given to a deformed bar, because the deformed bar continues to hang on long after the amount of deformation has taken place at which the beam reinforced with such a bar would have failed.

On the other hand, the beam automatically gives the criterion for load at which the results should be compared. You increase the bond resistance so that the beam will carry more load; therefore the amount of load that will be carried by the beam is the criterion for the comparison of bars or quality of concrete, or whatever the comparison is based on. Besides, the case in which the bond is usually important is that of a flexural member. There are not many members in which only direct tension or compression is involved in which bond is extremely important. The beam therefore not only furnishes the best criterion for comparison of bond resistances but it is the form of member to which the results of tests are most often applied. A good many beam tests have been made for bond, but it is not entirely clear just how the load should be applied. Tests have been made in which the load which caused the first slip, if maintained on the beam for a long time, caused failure of the beam without any increase in the load; yet with similar beams loaded rapidly to failure a very much higher load has been carried. Therefore it seems that the kind of test which should be made is a test in which a beam is used, and in which the load is maintained for a considerable length of time. Some suggestions for the design of a test beam and its loading to determine bond resistance are given by the writer in an article in *Engineering News-Record* for June 25, 1925, discussing certain European tests.

Mr. Abrams. MR. D. A. ABRAMS[1].—In the report[2] referred to by Mr. Kelley, published some fifteen years ago, there were quite a number of tests of the type in which a pull-out load was applied to specimens cast in horizontal position; however, the bulk of the specimens was molded in a vertical position. Those tests showed that the bond developed by the

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[1] Director of Research, International Cement Corporation, New York City.

[2] D. A. Abrams, "Tests of Bond Between Concrete and Steel," *Bulletin No. 71*, Engineering Experiment Station, University of Illinois, 1913.

Mr. Abrams. horizontally molded specimen was about 60 per cent of that of specimens molded in a vertical position. We found that the bond developed in a horizontal position was extremely sensitive to the restraint of the bar; if the bar was free to subside with the concrete, as it generally is in a beam or girder, the bond strength was not materially reduced; however, if the bar was held rigidly we found a serious reduction in strength and the hollow, watery spaces under the bar, as pointed out by Mr. Edwards.

Mr. Slater spoke of the desirability of making bond tests on beams. If you are interested in bond, *per se*, the easiest and most direct method of testing is in a pull-out specimen molded in a vertical position. If you are interested in the freakish results which may be produced by more or less uncertain conditions of bar subsidence, etc., you will get that in beam tests. In using beams for bond tests, you are simply setting up a very laborious, complicated, and indirect method of getting something which is in itself quite simple.

Mr. Richart. MR. F. E. RICHART[1].—I should like to add to what Mr. Abrams has said that I think a good type of bond test specimen would be a beam with the reinforcement bar cast near the top surface, as it would be placed over the support of a restrained beam. I do not think that such specimens have ever been used in bond tests. They have been used, as Mr. Slater has mentioned, in test beams designed to fail by diagonal tension; in several of these beams the failure was primarily by bond due to the fact that the concrete had settled away from the reinforcement bars in a manner similar to that described by Mr. Edwards.

The idea of defective bond on the lower side of the reinforcement bar may well be extended to the bond conditions in concrete between the lower side of a coarse aggregate particle and the adjacent mortar matrix, concerning which Mr. Edwards made certain observations in a paper before this Society in 1918[2]. Undoubtedly this matter of bond has a bearing on the paper presented at this annual meeting by Messrs. Gonnerman and Shuman[3]. I think it is worth remembering that concrete as usually poured is not homogeneous, but that as the mass settles and the water, laitance and air bubbles rise, there is a tendency for the latter to collect beneath the larger particles of aggregate, thus producing numerous but unconnected surfaces of weakness in more or less horizontal planes throughout the material. In a wet mixture these planes may be expected to be more pronounced than in a dry one.

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[1] Research Associate Professor of Theoretical and Applied Mechanics, Engineering Experiment Station, Laboratory of Applied Mechanics, University of Illinois, Urbana, Ill.

[2] L. N. Edwards, "Effects of Grading of Sands and Consistency of Mix upon the Strength of Plain and Reinforced Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. XVIII, Part I, p. 304 (1918).

[3] H. F. Gonnerman and E. C. Shuman, "Compression, Flexure and Tension Tests of Plain Concrete," see p. 527.

Mr. Richart.

The planes of weakness evidently have a distinct bearing upon tests of concrete in tension, compression and flexure. In the report for 1925 of Sub-Committee III on Field Tests for Concrete, of Committee C-9 on Concrete and Concrete Aggregates,[1] it was stated that compression test cubes when tested in the position cast showed on the average 13 per cent greater strength than companion specimens tested with the load applied at right angles to the direction of settlement during pouring. This is consistent with the idea that failure in a compression test is precipitated by splitting within the concrete on planes parallel to the direction of loading. When the specimen is placed so that the splitting may occur along its planes of weakness, failure is hastened.

Tests have been made on transverse specimens showing that the specimen cast vertically and tested on its side is somewhat weaker than the specimen poured and tested on its side, a result that also bears out the idea of planes of weakness in the material. I think most of you have observed that if you break a test cylinder in two, particularly if it contains smooth pebble aggregates, the pebbles at the fracture will adhere to the part of the cylinder that was uppermost when poured. This effect should be seen in the tension test. The tension specimens used by Gonnerman and Shuman were poured in a vertical position and the settlement planes were therefore in the position to do the most damage in producing tensile failure, whereas if the specimens had been cast horizontally, the relation of the tensile strength to the transverse and compressive strengths evidently would have differed somewhat.

Mr. Derleth.

MR. C. P. DERLETH[2].—Segregation of the concrete components primarily is the cause of the water film on the under side of reinforcement bars horizontally placed. In fireproofing construction it causes water pockets on the under side of beams. Segregation is induced by the fact that in fireproofing as well as in pouring concrete around reinforcing steel, wet consistencies are used and consequently the coarse aggregate is pulled down by force of gravity displacing upwardly the lighter constituent of the concrete, the water. There was a paper along this line presented by Herbert J. Gilkey at the Chicago meeting of the American Concrete Institute[3] which brought out this same effect of water pocketing on the under side of coarse aggregates deleteriously influencing bond.

Field practice in fireproofing construction of using wire rein-

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part I, p. 199 (1925).

[2] Colloy Products Co., St. Louis, Mo.

[3] Herbert J. Gilkey, "Coarse Aggregate in Concrete as a Field for Needed Research," *Proceedings*, Am. Concrete Inst., Vol. XXIII, p. 363 (1927).

Mr. Derleth. forcement on the under side of structural beams to give firm attachment of concrete to steel is evidence of lack of bond due to segregation. Suggested ideas on further research on this problem would be through the determination of the effect of variations in fineness or size grain distribution of the various cements in offsetting this inevitable segregation had in plain concrete. The effect on bond of gel or colloidal types of admixtures which have ideal water retention and suspending capacity, this due to their effect of increasing viscosity of mortar without reduction in workability, would justify study.

Mr. Edwards. Mr. L. N. EDWARDS[1] (*author's closure by letter*).—In so far as I have a knowledge the concrete-steel bond tests reported by other investigators have not included variations in the water content of the concrete mixes nor have they individually involved the use of different sands. Concerning the reference made by Mr. Kelley to the Joint Committee recommendation it is not entirely clear that 4 per cent of the compressive strength of the concrete will provide an ample margin of safety when actual construction conditions are considered. The test results indicate a possibility that concrete mixes rich in cement and therefore likely to produce high-strength concretes may not produce correspondingly high bond strengths. Sands in which the "Mother rock" origin and the granulometric composition are conducive to the production of high-strength mortars and concretes, other conditions being equal, will unquestionably produce concretes having higher compressive strengths than will sands having less valuable mortar and concrete-making properties, but the tests here under consideration do not give assurance that bond upon steel will vary accordingly. As a matter of fact, concrete-steel bond appears to be dependent upon and influenced by various factors affecting not only the concrete materials but also the making, placing, curing, etc., of the finished concrete product.

The remarks of Mr. Richart and Mr. Derleth upon the tendency of the water content of a concrete mix to accumulate beneath the reinforcement bars and the stone and sand particles recalls an investigation made in 1917 which was described in a short article in *The Canadian Engineer*.[2] The final result of the investigation indicated that cement paste will retain a quantity of water approximately 4 per cent in excess of that required to produce a standard normal consistency neat cement test mix. The segregation of the water content of a concrete is, therefore, to be expected during the process of placing it in the forms.

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[1] Bridge Engineer, State Highway Commission, Augusta, Me.

[2] *The Canadian Engineer* (Toronto) May 15, 1919, Vol. 36, p. 449.

THE EFFECT OF STRENGTH OF BRICK ON COMPRESSIVE STRENGTH OF BRICK MASONRY[1]

By J. W. McBurney[2]

Synopsis

The paper reports the data secured in the testing of 186 brick walls where five different makes of brick were used. The five makes of brick were tested in compression flatwise and edgewise, in transverse bending, in tension and in shear. Ratios of these various brick strengths to both maximum compressive strength and strength at first crack of the walls were calculated.

Comparison of ratios where workmanship, strength of mortar and type of construction were the same indicate that the compressive strength of brick tested flatwise gives the most uniform relation to strength of masonry.

Introduction

It is recognized that the strength of the individual brick unit is one of the most important factors in determining the compressive strength of brick masonry. However, opinions differ as to which one of the strength properties of the bricks is the controlling factor in the compressive strength of brick masonry. Those interested in the records of earlier workers will find a rather complete bibliography in the appendix of the Building Code Committee's "Recommended Minimum Requirements for Masonry Wall Construction."[3]

The majority of these investigators put the relationship in the general terms of "strength of brick" or give their approval to compressive strength (usually measured flatwise) as the best measure of the compressive strength of the masonry. However, Boyd[4] states "the modulus of rupture usually determines the failing point of a wall or pier." Emley and Cleare[5] recommend that the tensile strength of the brick be accepted as the criterion of pier strength. These quotations show a difference of opinion and warrant the introduction of some new data.

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[1] Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Research Associate of the Common Brick Manufacturers' Association of America at the U. S. Bureau of Standards, Washington, D. C.

[3] "Recommended Minimum Requirements for Masonry Wall Construction," U. S. Department of Commerce Elimination of Waste Series, Report of Building Code Committee (1925).

[4] D. K. Boyd, "Brick," *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part II, p. 315 (1925.)

[5] W. E. Emley and W. M. Cleare, "How Brick Piers Fail," *Proceedings*, Sand-Lime Brick Association (1920).

SOURCE OF DATA

The data presented in this paper were secured in part during the investigation of the compressive strength of brick masonry conducted by the U. S. Bureau of Standards in cooperation with the Common Brick Manufacturers' Association of America during 1926–1927. This work included building and testing in compression 168 walls, 6 ft. long by 9 ft. high, and 129 small walls or wallettes 18 in. long by 34 in. high. Use is made of data presented by Whittemore and Stang[1] on the compressive strength of 18 walls and 18 wallettes built of sand-lime brick.

These two sources of data provide a total of 186 walls and 147 wallettes. Of these 186 walls, 93 were 8-in. or 12-in. solid construction laid up in common American bond with headers every sixth course. The other 93 included twelve 4-in. walls with pilasters and 81 hollow walls built in whole or in part with brick on edge. For a description of the methods of laying these hollow walls, reference should be made to a publication of the Common Brick Manufacturers' Association.[2] These 186 walls represent five makes of brick, three mixes of mortar,[3] ten types of construction and two grades of workmanship.

As this paper, except for a brief review of the effect on compressive strength of variations in other factors than brick, is confined to the effect of strength of brick on compressive strength of masonry, a selection has been made from these 186 wall tests and 147 wallette tests so that a direct comparison is afforded between walls and wallettes where the workmanship, construction and mortar are comparable and where the only recognized variable is the brick.

Five kinds of brick were used in the tests herein reported. A brief description follows:

A—Stiff-mud, end cut, double column from Chicago district. Irregular in shape and quite laminated.

E—Soft-mud brick from Detroit. These brick are manufactured with a depression or frog in one side 0.4 in. in depth.

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[1] H. L. Whittemore and A. H. Stang, "Compressive Strength of Sand-Lime Brick Walls," U. S. Bureau of Standards *Technologic Paper No. 276* (1925).

[2] "Hollow Walls of Brick and How to Build Them," Common Brick Manufacturers' Association of America, Cleveland, Ohio.

[3] Actually five mixes of mortar were used. But for the purpose of this paper the $1:1\frac{1}{2}:6$ cement-lime mortar is considered as the equivalent of the $1:1:6$ cement-lime mortar and the $1:3$ cement mortar is regarded as having the same effect on the strength of masonry as a $1:\frac{1}{10}:3$ cement-lime mix.

H—Soft-mud, sand struck from New Hampshire. Rather irregular in shape and size.

I—Dry-pressed brick from Mississippi. Very regular in size and with plane and parallel surfaces.

J—Sand-lime brick from Ohio. Smooth and regular.

Methods of Testing and Sampling

Tests of Bricks:

The bricks were tested in compression flatwise, compression edgewise, flexure, tension, and in punching shear.

Tests in compression were made in accordance with the Tentative Methods of Testing Brick (C 67 – 27 T) of the American Society for Testing Materials.[1] The half-bricks were from the flexure test, from the tension test, or were cut by chisel for this test. No significant difference was observed; hence the average of all tests was used. The transverse or flexural tests were made on whole bricks flatwise. Some of the tests were made on the standard testing machine with knife edges as described in the above-mentioned A.S.T.M. methods, and some on the equalizer apparatus described by Whittemore.[2] No significant difference being found, the averages were used. The tension test was made by means of an apparatus designed for the purpose.[3] Shearing tests were made with an apparatus designed by H. H. Dutton for use in testing limestone.[4]

Sampling of Bricks:

The method of sampling may be of interest. Each shipment of brick, from 9000 to 52,000 bricks, was sampled, after delivery, at 50 points near the surface of the pile. Twenty bricks were taken from each place, some from as much below the surface as was practicable without displacing many of the brick on the surface of the pile. The first group of 20 was then laid in a row, end to end, succeeding groups were laid alongside of the first and later on top of those previously placed. The result was a rectangular pile of bricks from the end of which 50 bricks were taken. These fairly represented each portion of the surface of the original heap. Fifty was the unit of sampling.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 826 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 269.

[2] H. L. Whittemore, "Equalizer Apparatus for Transverse Tests of Brick," U. S. Bureau of Standards *Technologic Paper No. 251* (1924).

[3] J. W. McBurney, "Strength of Brick in Tension," *Journal*, Am. Ceramic Soc., Vol. 11, No. 2, p. 114, February, 1928.

[4] D. W. Kessler and W. H. Sligh, "Physical Properties of the Principal Commercial Limestone Used for Building Construction in the United States," U. S. Bureau of Standards *Technologic Paper No. 349*, p. 508 (1928).

TABLE I.—STRENGTHS OF BRICK.

Brick	Compressive Strength				Transverse Strength		Tensile Strength		Shearing Strength	
	Half Brick Flatwise		Half Brick on Edge							
	Number of Tests	Average, lb. per sq. in.	Number of Tests	Average, lb. per sq. in.	Number of Tests	Modulus of Rupture, lb. per sq. in.	Number of Tests	Average, lb. per sq. in.	Number of Tests	Average, lb. per sq. in.
A	94	3370	100	3 780	100	1320	50	420	10	2200
E	50	3320	50	3 360	97	670	50	220	10	2330
H	100	8610	50	11 600	100	1550	50	600	4[b]	7100
I	98	3520	50	3 630	97	800	50	320	10	3180
J	50[a]	4150	50[a]	2 760	50[a]	560	50[a]	210	70[c]	1230

[a] Whittemore and Stang, U. S. Bureau of Standards *Technologic Paper No. 276.*
[b] Slabs 1 in. in thickness.
[c] Unpublished data of H. V. Johnson. See remarks in text.

TABLE II.—DISTRIBUTION OF INDIVIDUAL TESTS OF BRICK: COMPRESSIVE STRENGTH, FLATWISE.

Compressive Strength Range, lb. per sq. in.	Number of Tests				
	Brick A	Brick E	Brick H	Brick I	Brick J
1 500 – 1 999	..	..	..	..	..
2 000 – 2 499	5	1	..	6	..
2 500 – 2 999	16	18	..	28	8
3 000 – 3 499	40	14		29	3
3 500 – 3 999	21	11	1	10	10
4 000 – 4 499	7	2	..	8	11
4 500 – 4 999	4	4	..	6	8
5 000 – 5 499	1	..	3	4	6
5 500 – 5 999	..	..	3	1	2
6 000 – 6 499	..	..	2	3	1
6 500 – 6 999	..	..	11	1	..
7 000 – 7 499	..	..	10	..	..
7 500 – 7 999	..	..	9	1	1
8 000 – 8 499	..	..	9	..	..
8 500 – 8 999	..	..	10	..	..
9 000 – 9 499	..	..	16	..	..
9 500 – 9 999	..	..	5	..	..
10 000 – 10 499	..	..	8	..	..
10 500 – 10 999	..	..	5	..	..
11 000 – 11 499	..	..	4	..	..
11 500 – 11 999	..	..	3	..	..
12 000 – 12 499	..	..	1	..	..
Total Number of Tests	94	50	100	98	50

TABLE III.—DISTRIBUTION OF INDIVIDUAL TESTS OF BRICK: MODULUS OF RUPTURE.

Transverse Strength Range, lb. per sq. in.	Number of Tests				
	Brick A	Brick E	Brick H	Brick I	Brick J
0 – 199	..	2	..	..	..
200 – 399	1	1	..	2	3
400 – 599	7	25	..	15	29
600 – 799	9	53	..	43	16
800 – 999	11	10	6	24	2
1000 – 1199	15	6	11	12	..
1200 – 1399	16	..	19	2	..
1400 – 1599	9	..	20	2	..
1600 – 1799	12	..	17	..	..
1800 – 1999	9	..	13	..	..
2000 – 2199	8	..	4	..	..
2200 – 2399	1	..	4	..	..
2400 – 2599	2	..	2	..	..
2600 – 2799	..	..	1	..	..
Total Number of Tests	100	97	97	100	50

When in Table I numbers other than 50 or multiples of 50 occur, it is due to loss or breakage of samples in testing.

Tests of Masonry:

For methods of testing walls and wallettes reference is made to Whittemore and Stang's report.[1]

In Table I are presented the averages of the compressive strength flatwise, compressive strength edgewise, transverse strength (modulus of rupture), tensile strength and shearing strength of the five makes of brick.

Attention should be called to the value for shearing strength reported for brick *J*. The original value (from Mr. Johnson) was not obtained from a random sample, as were all the other values given in this table. All Johnson's results were consistently low compared with Whittemore and Stang's values. Hence, it was considered justifiable to raise his figure from 1055 lb. per sq. in. as reported, to the 1230 lb. per sq. in. given in the table. This was done according to the ratio of compressive strength on edge (Johnson), 2364 lb. per sq. in., to Whittemore and Stang's value, 2760 lb. per sq. in.

Tables II and III give the distribution of results from tests for compressive strength flatwise and transverse strength (modulus of rupture), averages of which appear in Table I.

Tables IV and V give compressive strength at first crack and maximum strength for a selection of solid walls and wallettes. In general, the average for a set of six walls represents the average of results with three 8-in. and three 12-in. walls that were considered comparable.

FACTORS GOVERNING THE COMPRESSIVE STRENGTH OF BRICK MASONRY OTHER THAN STRENGTH OF BRICK

Before entering upon an analysis of the data presented in these tables, a brief review of the effect on strength of brick masonry of factors other than strength of brick may be in order. The following factors may be considered as affecting the compressive strength of masonry:

1. Workmanship;
2. Type of Construction;
3. Strength of Mortar;
4. Adhesion of Brick to Mortar; and
5. Uniformity and Regularity of Size and Shape of Brick Unit.

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[1] H. L. Whittemore and A. H. Stang, "Compressive Strength of Sand-Lime Brick Walls," U. S. Bureau of Standards *Technologic Paper No. 276* (1925).

A number of other factors can be listed. They are, however, considered as subdivisions of the factors here listed.

TABLE IV.—COMPRESSIVE STRENGTH OF WALLS.

Group	Brick	Number of Walls Tested	Load at First Crack, lb. per sq. in.			Maximum Load, lb. per sq. in.			Ratio, Load at First Crack to Maximum Load
			Average	Maximum	Minimum	Average	Maximum	Minimum	
8 AND 12-IN. SOLID WALLS. MASON B. 1:3 CEMENT AND 1:$\frac{1}{10}$:3 CEMENT MORTAR									
No. 1........	A	10	908	1165	706	954	1200	756	0.951
	E	6	795	1038	586	1145	1345	893	0.69
	H	6	2028	2525	1344	2710	3228	2040	0.75
	I	6	1330	1763	700	1510	1855	1333	0.88
8 AND 12-IN. SOLID WALLS. MASON A. 1:3 CEMENT MORTAR									
No. 2........	A	5	624	709	570	661	751	578	0.94
	H	1	1500	...	...	2032	....	...	0.74
	I	1	870	...	...	870	....	...	1.00
	J	6	700	800	550	905	1030	800	0.77
8 AND 12-IN. SOLID WALLS. MASON B. 1:1:6 CEMENT-LIME MORTAR									
No. 3........	E	6	553	650	400	946	1062	760	0.59
	H	6	1134	1500	900	1840	1991	1710	0.62
	I	6	1140	1374	900	1232	1374	967	0.92
8 AND 12-IN. SOLID WALLS. MASON A. 1:1$\frac{1}{4}$:6 CEMENT-LIME MORTAR									
No. 4........	A	6	515	585	400	586	618	574	0.88
	J	5	510	550	400	638	770	550	0.80
MASON A. 1$\frac{1}{4}$:3 LIME MORTAR									
No. 5........	A	6	203	250	150	286	352	249	0.71
	J	6	208	250	200	302	320	280	0.69
"ECONOMY" 4-IN. SOLID WALLS. MASON B. 1:$\frac{1}{10}$:3 CEMENT MORTAR									
No. 6........	H	3	2274	2400	2091	3146	3520	2757	0.72
	I	3	1575	1877	1200	1624	1877	1348	0.97
"ECONOMY" 4-IN. SOLID WALLS. MASON B. 1:1:6 CEMENT-LIME MORTAR									
No. 7........	H	3	1233	1400	1100	1938	1974	1879	0.64
	I	3	1373	1510	1238	1433	1564	1370	0.96

Workmanship:

Reference should be made to another paper[1] for a detailed study, based upon these same data, of the effect of workmanship on strength of masonry. The principal conclusions are quoted from this paper:

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[1] J. W. McBurney, "Effect of Workmanship on Strength of Brick Masonry," *American Architect*, Vol. CXXXII, No. 2532, p. 613, November 5, 1927.

Workmanship characterized by filled vertical joints and unfurrowed horizontal beds will give an increase of 24 to 112 per cent wall strength over the results of workmanship characterized by unfilled vertical joints and furrowed horizontal bed.

All other things being equal, the thinner the mortar joint the stronger the masonry for solid brick walls.

The percentage increase in strength of solid walls secured by using filled and unfurrowed joint workmanship over the other type diminishes with increase in strength of the brick.

Table V.—Compressive Strength of Wallettes.

Group	Brick	Number of Tests	Load at First Crack, lb. per sq. in.			Maximum Load, lb. per sq. in.			Ratio of Load at First Crack to Maximum Load
			Average	Maximum	Minimum	Average	Maximum	Minimum	
Mason B. $1:\frac{1}{10}:3$ Cement Mortar									
No. 1	E	6	1205	1550	592	1344	1674	783	0.897
	H	6	2140	3085	1233	3530	4139	2516	0.606
	I	6	1506	1750	1389	1696	2058	1501	0.888
Mason A. 1:3 Cement Mortar									
No. 2	A	5	675	877	409	775	884	701	0.871
	J	6	900	1000	800	1185	1380	1070	0.759
Mason B. 1:1:6 Cement-Lime Mortar									
No. 3	E	6	780	1202	442	1033	1313	704	0.755
	H	6	1516	1875	1032	2380	2847	1728	0.637
	I	6	1219	1765	672	1344	1793	1018	0.907
Mason A. $1:1\frac{1}{4}:6$ Cement-Lime Mortar									
No. 4	A	6	564	710	375	685	736	624	0.824
	J	6	633	700	550	875	1040	740	0.724
Mason A. $1:1\frac{1}{4}:3$ Lime Mortar									
No. 5	A	6	242	321	137	390	419	351	0.621
	J	6	258	300	250	420	470	360	0.615

Type of Construction:

No light is thrown on the effect of increasing or decreasing the number of header courses by the present investigation. All solid construction was of the same type. It might be noted that there is a decided lack of unanimity in the conclusions reached by the several investigators who have reported on the effect of varying the masonry bond. Reference should be made to Kreuger[1] and Bragg[2] for data and discussion. However, data are afforded for the comparison of the various types of hollow walls with solid construction, as well as the effect of size as shown by the strength of comparable walls and wallettes. Stang[1] covers the comparison afforded by these data in another paper. For brick of lot *I* of the present series a very excellent agreement is found between the compressive strengths of hollow and solid walls of similar mortar and workmanship on the basis of the net cross-sections of the various types.

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[1] H. Krueger, "Die Festigkeit des Ziegelmauerwerks," *Tonindustrie Zeitung*, Vol. 40, p. 597 *et seq.*, September 9–21 (1916) and, in English, *Clay Worker*, Vol. 68, pp. 42 and 46, July and August, 1917.

[2] J. G. Bragg, "Compressive Strength of Large Brick Piers," U. S. Bureau of Standards *Technologic Paper No. 111* (1918).

Wallettes are quite generally stronger than the corresponding walls. For the sand-lime series, the walls range from 60 per cent to 80 per cent of the wallette strength, with an average for all mortars of 73 per cent. Comparison of the solid walls and wallettes of the later series gives values for the walls of 60 per cent up. A few walls were stronger than the corresponding wallettes. The average strength of wall with relation to the corresponding wallette strength for all solid walls, excluding those of sand-lime brick, is 78 per cent. Hollow wallettes will frequently give 100 per cent greater strength than the corresponding walls, due probably to relatively small slenderness ratios. It will be noted that both walls and wallettes give values for slenderness ratio (length divided by radius of gyration) well under twenty when the dimensions of the wall proper are considered. However, when the headers or bond joining the wythes are broken, in the case of a hollow wall, there results an unsupported section 9 ft. high by $2\frac{1}{4}$ in. thick, which is well within the range of column action.

Strength of Mortar:

By strength of mortar is meant the compressive strength of cylinders 2 in. in diameter by 4 in. in length made from the batch or batches of mortar used in the wall. These cylinders were cured some in air and some in water. The strength of mortar is affected originally by the percentage of sand, lime and cement used. This we can control. Then water is added to the taste of the brick mason. Finally, the condition of curing is set by such factors as water content, total absorption, rate of absorption of the brick and after-treatment of the wall. The "true compressive strength" of mortar when the test specimen has the form and dimensions of a typical mortar joint is a matter of conjecture.

Except where lime mortar was used, the typical failure of brick masonry at maximum load was by crushing of brick. Mortar 1:1:6 or stronger was apparently uninjured. Regardless of whether mortar is stronger than brick, there is the fact that increase in mortar strength is associated with increase in masonry strength, other factors remaining unchanged. Contrary results reported by Bragg[1] and Macgregor[2] are probably explainable on the basis of variation in the workmanship factor.

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[1] A. H. Stang, "The Strength of Solid and of Hollow Walls of Brick," *Ceramic Age*, Vol. X, No. 6, p. 198, December, 1927; also *Architect and Builder*, Vol. 4, No. 7, p. 141, February 15, 1928.

The following results are the averages of some 1000 cylinders tested as part of the present investigation on strength of masonry:

Mixture (by volume)	Compressive Strength, lb. per sq. in. Cured in Water	Compressive Strength, lb. per sq. in. Cured in Air
$1\frac{1}{4}$ Lime : 3 Sand	....	90
1 Cement : $1\frac{1}{4}$ Lime : 6 Sand	1070	500
1 Cement : 1 Lime : 6 Sand	1100	750
1 Cement : $\frac{1}{10}$ Lime : 3 Sand	3260	1950
1 Cement : 3 Sand	3580	1460

Using these values and comparing with wall strengths where brick, workmanship and construction are the same, the most constant ratio is that of the cube root of the mortar strength to the corresponding masonry strength. This is a rough approximation and doubtless can be improved. Evidence will be presented later pointing to an effect on ratios of brick strength to masonry strength by changes in mortar strength, and it is reasonable to assume that the ratios of mortar strength to wall strength would also be influenced by changing brick strength.

Adhesion of Mortar to Brick:

Adhesion of mortar to brick has been suggested as very important in determining the strength of brick masonry. Through the courtesy of Mr. H. V. Johnson[3] the following unpublished results are made available representing the strength in tension developed between two bricks laid flat with a mortar having the composition of 75 per cent of sand and 25 per cent of portland cement by weight and 23 per cent of added water:

Brick	*A*	*E*	*H*	*I*	*J*
Strength of Joint in Tension, lb. per sq. in.	79.3	58.6	76.0	51.4 (87.5 wet)	18.0–38.2

The bricks were laid dry. Each figure is an average of ten tests. The value in parenthesis for brick of lot *I* was on the same 20 bricks reversed and wetted. Mr. Johnson reports that the individual test values are fairly well related to the rate of absorption of the individual bricks. Failures took place sometimes within the joints; more often at the union of the mortar with one of the brick faces, and a very few bricks sheared out of the jaws of the machine. The water absorption of all the bricks used in these tests had been previously determined by boiling. It is possible that the boiling may have affected the surface condition of the specimens in some manner.

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[1] J. S. Bragg, "Compressive Strength of Large Brick Piers," U. S. Bureau of Standards *Technologic Paper No. 111* (1918).

[2] J. S. Macgregor, *Bulletin 300*, National Lime Association, First Edition, p. 6 (1920).

[3] Junior Chemist, U. S. Bureau of Standards, Washington, D. C.

Comparison of the results of these few tests with the compressive strengths of the masonry given in Tables IV and V shows no apparent relation of the adhesion of the brick and the mortar to the compressive strength of the masonry. If it were a question of the masonry acting as a beam, it is probable that this factor would be of the first importance.

TABLE VI.—RATIOS OF VARIOUS BRICK STRENGTHS

Brick	Ratio, Compressive Strength Flat to Compressive Strength on Edge	Ratio, Modulus of Rupture to Compressive Flatwise Strength	Ratio, Tensile Strength to Modulus of Rupture	Ratio, Tensile Strength to Compressive Flatwise Strength	Ratio, Shearing Strength to Compressive Flatwise Strength	Ratio Shearing Strength to Modulus of Rupture
A	0.89	0.39	0.32	0.12	0.65	1.7
E	0.99	0.20	0.33	0.067	0.70	3.5
H	0.74	0.18	0.39	0.070	0.87	4.6
I	0.97	0.23	0.40	0.090	0.90	4.0
J	1.50	0.13	0.38	0.051	0.30	2.2
Mean Deviation divided by Mean	0.20	0.30	0.082	0.25	0.25	0.32

Uniformity and Regularity of Size and Shape of Brick Units:

Uniformity of size and shape is the ever present help in trouble, and of assistance when all other factors fail. However, consider a simple case. A brick supported irregularly on a mortar bed breaks under a given load. Consider that the failure is that of the brick acting as a beam. The maximum load can be increased by three different procedures. First, a stronger brick can be used; second, by improved workmanship the arching or bridging can be reduced to a uniform bearing; and third, in so far as the arching or bridging effect is due to the brick, using a regular shaped unit having plane parallel faces will add to the apparent strength. Where there is a pronounced difference in strength and modulus of elasticity between the brick and mortar, actual bearing may be present but localized stresses certainly will occur if there is pronounced irregularity in size or shape. Testing bricks in compression by the A.S.T.M. procedure, of course, suppresses this factor. Some interesting comparisons might be made by testing uncapped bricks in compression. The influence of variation in strength of the individual bricks above and below their mean is discussed in the next section.

EFFECT OF STRENGTH OF BRICK ON COMPRESSIVE STRENGTH OF MASONRY

Returning now to the subject of the paper, the first question requiring an answer deals with the possibility that there may be such a constant and regular relation between the various measures of strength that one measure will serve as well as another in fixing the strength of brick. Table VI derived from Table I should throw some light on this.

Ratios of Brick Strengths:

Consider first the ratio, compressive strength flatwise to compressive strength edgewise. Bricks *A*, *E* and *I* can be considered as giving the same result when tested flatwise and on edge. Brick of lot *H* is consistently 25 per cent weaker tested flatwise than when tested on edge. No explanation is offered; it is simply an observation. Brick *J*, like most sand-lime and cement bricks, is stronger tested flatwise than edgewise. We may conclude, then, that the results of compression tests flatwise and edgewise give various ratios for various bricks.

Consider next the ratio, modulus of rupture to compressive strength flatwise. Brick *A* has a ratio almost 100 per cent above the average of all the others. Very fair agreement exists for the three soft-mud and dry-pressed bricks. Brick of lot *J* is low. The conclusion is that if discussion is confined to soft-mud and dry-pressed bricks, modulus of rupture and compressive strength flatwise could be used interchangeably, because one is the measure of the other. But the ratio does not hold for other materials and other methods of manufacture. It would be no occasion for surprise if some make of soft-mud or dry-pressed brick should on test give a ratio for modulus of rupture to compressive strength flatwise far from 0.20. Two tests are sufficient to show a difference. A very great many tests are needed before uniformity can be regarded as demonstrated.

The ratio, tensile strength to modulus of rupture, is reasonably uniform for all five bricks.

The ratio, shear to compressive strength flatwise, is rather variable and is lowest for brick *J*. The ratio, shear to modulus of rupture, is a rather irregular ratio.

In general then on the basis of these data, the relation between tensile and transverse strength is the only one sufficiently uniform to warrant taking one kind of strength as the measure of another kind of strength for all makes of brick.

A brief consideration of the properties of these bricks and the nature of the tests may throw some light on the variations of these ratios. Brick, lot *A*, due to its laminar structure and stiff-mud end-cut manufacture, gives the appearance of a bundle of fibers. A high modulus of rupture would be expected. A brick, equally laminated but side-cut, all other things being equal, would probably give a very much lower value for modulus of rupture. By theory, direct stresses of compression produce shearing stresses and strains in all planes in the interior of a body except those perpendicular and parallel to the line of application of stress. Obviously, some of these may coincide with the planes or rather surfaces of weakness due to laminated structure. The same considerations apply to shearing and tensile strength.

Bricks *E*, *H*, and *I*, all soft-mud or dry-pressed, show no very marked divergence among themselves in their strength ratios. The chief anomaly requiring explanation (which is lacking) is the higher compressive strength edgewise of brick *H*.

Brick *J* appears to lack "cohesion." Shearing, transverse and tensile strengths are low. For brick *J* low shearing strength seems the explanation of the pronounced effect on compressive strength produced by varying the ratio of height to cross-section.

Ratios of Wall Strengths to Brick Strengths:

In Tables VII, VIII, IX and X, ratios of wall and wallette strengths to the five measures of brick strength are given. These ratios are derived from the data of Tables I, IV and V. Each group of values have all factors, workmanship, mortar and type of construction constant. The brick is the variable. In addition to the average, the ratio of the maximum and minimum wall strength of each group to the brick strength is given. It was hoped that some regular relation would be evident from the individual maximum and minimum values. Such relation is not apparent, but these values at least give a measure of the consistency of the tests.

In the analysis of these values the following two assumptions are made: (1) for a set of tests where workmanship, mortar, and type of construction are the same, that strength of brick is considered the best measure of masonry strength which gives the most nearly constant ratios of wall strength to brick strength; and (2) where other factors such as mortar, proportion and workmanship are changed and change appears in the ratios of the wall strength to brick strength ratios, it is assumed that this variation is caused by the effect of

TABLE VII.—RATIOS OF COMPRESSIVE STRENGTH OF WALLS TO STRENGTHS OF BRICK.

Group	Brick	Number of Wall Tests	Ratio, Wall Strength to Compressive Strength of Brick Flatwise			Ratio, Wall Strength to Compressive Strength of Brick on Edge			Ratio, Wall Strength to Modulus of Rupture			Ratio, Wall Strength to Tensile Strength of Brick			Ratio, Wall Strength to Shearing Strength of Brick		
			Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
8 AND 12-IN. SOLID WALLS. MASON B. 1:3 AND $1:\frac{1}{10}:3$ CEMENT MORTAR																	
No. 1.	A	10	0.28	0.36	0.22	0.25	0.32	0.20	0.72	0.91	0.57	2.3	2.9	1.8	0.43	0.54	0.34
	E	6	0.34	0.40	0.27	0.34	0.40	0.27	1.71	2.00	1.33	5.2	6.1	4.1	0.49	0.58	0.38
	H	6	0.31	0.37	0.24	0.23	0.28	0.18	1.75	2.08	1.32	4.5	5.4	3.4	0.38	0.45	0.29
	I	6	0.43	0.53	0.38	0.42	0.51	0.37	1.89	2.33	1.67	4.7	5.8	4.2	0.47	0.58	0.42
Mean Deviation divided by Mean..			0.13	0.14	0.18	0.23	0.21	0.25	0.26	0.25	0.27	0.21	0.21	0.23	0.09	0.07	0.11
8 AND 12-IN. SOLID WALLS. MASON A. 1:3 CEMENT MORTAR																	
No. 2.	A	5	0.20	0.22	0.17	0.17	0.20	0.15	0.50	0.57	0.44	1.6	1.8	1.4	0.30	0.34	0.26
	H	1	0.24	....	....	0.18	....	....	1.31	....	....	3.4	...	...	0.29	....	....
	I	1	0.25	....	....	0.24	....	....	1.09	....	....	2.7	...	...	0.27	....	....
	J	6	0.22	0.25	0.[illegible]9	0.33	0.37	0.29	1.62	1.84	1.43	4.3	4.9	3.8	0.74	0.84	0.65
Mean Deviation divided by Mean..			0.09	....	....	0.24	....	....	0.30	....	....	0.28	...	...	0.42	....	....
8 AND 12-IN. SOLID WALLS. MASON B. 1:1:6 CEMENT-LIME MORTAR																	
No. 3.	E	6	0.28	0.32	0.23	0.28	0.32	0.23	1.4	1.6	1.1	4.3	4.8	3.5	0.41	0.46	0.33
	H	6	0.21	0.23	0.20	0.16	0.17	0.15	1.2	1.3	1.1	3.1	3.3	2.8	0.26	0.28	0.24
	I	6	0.35	0.39	0.27	0.34	0.38	0.27	1.5	1.7	1.2	3.9	4.3	3.0	0.39	0.43	0.30
Mean Deviation divided by Mean..			0.18	0.19	0.09	0.27	0.28	0.20	0.07	0.10	0.03	0.11	0.12	0.10	0.17	0.18	0.10
8 AND 12-IN. SOLID WALLS. MASON A. $1:1\frac{1}{4}:6$ CEMENT-LIME MORTAR																	
No. 4.	A	6	0.17	0.18	0.17	0.16	0.16	0.15	0.44	0.47	0.43	1.4	1.5	1.4	0.27	0.28	0.26
	J	6	0.15	0.19	0.13	0.23	0.28	0.20	1.11	1.37	0.98	3.0	3.7	2.6	0.52	0.63	0.45
Mean Deviation divided by Mean..			0.06	0.03	0.13	0.18	0.27	0.14	0.43	0.49	0.39	0.36	0.42	0.30	0.32	0.38	0.27
8 AND 12-IN. SOLID WALLS. MASON A. $1\frac{1}{4}:3$ LIME MORTAR																	
No. 5.	A	6	0.085	0.105	0.074	0.076	0.093	0.066	0.22	0.27	0.19	0.68	0.84	0.59	0.13	0.16	0.11
	J	6	0.073	0.077	0.068	0.109	0.116	0.102	0.54	0.57	0.50	1.44	1.52	1.33	0.25	0.26	0.23
Mean Deviation divided by Mean..			0.08	0.15	0.04	0.18	0.11	0.21	0.42	0.36	0.45	0.36	0.29	0.39	0.32	0.24	0.35
"ECONOMY" 4-IN. SOLID WALLS. MASON B. $1:\frac{1}{10}:3$ CEMENT MORTAR																	
No. 6.	H	3	0.36	0.41	0.32	0.27	0.30	0.24	2.03	2.27	1.78	5.2	5.9	4.6	0.44	0.50	0.39
	I	3	0.46	0.53	0.38	0.45	0.52	0.37	2.04	2.35	1.69	5.1	5.9	4.2	0.51	0.59	0.42
Mean Deviation divided by Mean..			0.12	0.13	0.09	0.25	0.27	0.21	0.002	0.017	0.026	0.01	0.00	0.04	0.07	0.08	0.04
"ECONOMY" 4-IN. SOLID WALLS. MASON B. 1:1:6 CEMENT-LIME MORTAR																	
No. 7.	H	3	0.23	0.23	0.22	0.17	0.17	0.16	1.25	1.27	1.21	3.2	3.3	3.1	0.27	0.28	0.26
	I	3	0.41	0.44	0.39	0.39	0.43	0.38	1.79	1.96	1.71	4.5	4.9	4.3	0.45	0.49	0.43
Mean Deviation divided by Mean..			0.28	0.31	0.28	0.39	0.43	0.41	0.18	0.21	0.17	0.17	0.20	0.16	0.25	0.27	0.25

TABLE VIII.—RATIOS OF COMPRESSIVE STRENGTH OF WALLS AT FIRST CRACK TO STRENGTHS OF BRICK.

Group	Brick	Number of Wall Tests	Ratio, Wall Strength to Compressive Strength of Brick Flatwise			Ratio, Wall Strength to Compressive Strength of Brick on Edge			Ratio, Wall Strength to Modulus of Rupture			Ratio, Wall Strength to Tensile Strength of Brick			Ratio, Wall Strength to Shearing Strength of Brick		
			Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
8 AND 12-IN. SOLID WALLS. MASON B. 1:3 AND 1:$\frac{1}{10}$:3 CEMENT MORTAR																	
No. 1.	A	10	0.27	0.35	0.21	0.24	0.31	0.19	0.69	0.88	0.54	2.2	2.8	1.7	0.41	0.53	0.32
	E	6	0.24	0.31	0.18	0.24	0.31	0.17	1.19	1.55	0.87	3.6	4.7	2.7	0.34	0.45	0.25
	H	6	0.24	0.29	0.16	0.17	0.22	0.12	1.31	1.63	0.87	3.4	4.2	2.2	0.29	0.36	0.19
	I	6	0.38	0.50	0.20	0.37	0.49	0.19	1.67	2.20	0.88	4.2	5.5	2.2	0.42	0.55	0.22
Mean Deviation divided by Mean..			0.18	0.19	0.11	0.23	0.24	0.14	0.23	0.22	0.16	0.17	0.19	0.11	0.14	0.14	0.16
8 AND 12-IN. SOLID WALLS. MASON A. 1:3 CEMENT MORTAR																	
No. 2.	A	5	0.19	0.21	0.17	0.17	0.19	0.15	0.47	0.54	0.43	1.5	1.7	1.4	0.28	0.32	0.26
	H	1	0.17	....	....	0.13	....	....	0.97	....	....	2.5	...	...	0.21	....	....
	I	1	0.25	....	....	0.24	....	....	1.09	....	....	2.7	...	...	0.27	....	....
	J	6	0.17	0.19	0.13	0.25	0.29	0.20	1.25	1.43	0.98	3.3	3.8	2.6	0.57	0.65	0.45
Mean Deviation divided by Mean..			0.14	....	....	0.24	....	....	0.25	....	....	0.20	....	....	0.36	....	....
8 AND 12-IN. SOLID WALLS. MASON B. 1:1:6 CEMENT LIME-MORTAR																	
No. 3.	E	6	0.17	0.20	0.12	0.16	0.19	0.12	0.82	0.97	0.60	2.5	3.0	1.8	0.24	0.28	0.17
	H	6	0.13	0.17	0.10	0.10	0.13	0.08	0.73	0.97	0.58	1.9	2.5	1.5	0.16	0.21	0.13
	I	6	0.32	0.39	0.26	0.31	0.38	0.25	1.43	1.72	1.13	3.6	4.3	2.8	0.36	0.43	0.28
Mean Deviation divided by Mean..			0.36	0.28	0.42	0.44	0.42	0.45	0.29	0.27	0.31	0.24	0.21	0.25	0.28	0.26	0.32
8 AND 12-IN. SOLID WALLS. MASON A. 1:1¼:6 CEMENT-LIME MORTAR																	
No. 4.	A	6	0.15	0.17	0.12	0.14	0.16	0.11	0.39	0.44	0.30	1.2	1.4	0.95	0.23	0.27	0.18
	J	5	0.12	0.13	0.096	0.18	0.20	0.14	0.91	0.98	0.72	2.4	2.6	1.9	0.41	0.45	0.32
Mean Deviation divided by Mean..			0.11	0.13	0.11	0.12	0.11	0.12	0.40	0.38	0.41	0.33	0.30	0.33	0.28	0.25	0.28
8 AND 12-IN. SOLID WALLS. MASON A. 1¼:3 LIME MORTAR																	
No. 5.	A	6	0.060	0.074	0.044	0.054	0.066	0.040	0.15	0.19	0.11	0.48	0.60	0.36	0.092	0.11	0.068
	J	6	0.050	0.060	0.048	0.075	0.091	0.072	0.37	0.45	0.36	0.99	1.19	0.95	0.17	0.20	0.16
Mean Deviation divided by Mean..			0.09	0.10	0.04	0.16	0.16	0.29	0.42	0.41	0.53	0.35	0.33	0.45	0.30	0.29	0.40
"ECONOMY" 4-IN. WALL. MASON B. 1:$\frac{1}{10}$:3 CEMENT MORTAR																	
No. 6.	H	3	0.26	0.28	0.24	0.20	0.21	0.18	1.47	1.55	1.35	3.8	4.0	3.5	0.32	0.34	0.29
	I	3	0.45	0.53	0.34	0.43	0.52	0.33	1.97	2.35	1.50	4.9	5.9	3.8	0.50	0.59	0.38
Mean Deviation divided by Mean..			0.27	0.31	0.17	0.37	0.42	0.29	0.15	0.20	0.05	0.13	0.19	0.04	0.22	0.27	0.13
"ECONOMY" 4-IN. WALL. MASON B. 1:1:6 CEMENT-LIME MORTAR																	
No. 7.	H	3	0.14	0.16	0.13	0.11	0.12	0.095	0.80	0.90	0.71	2.0	2.3	1.8	0.17	0.20	0.15
	I	3	0.39	0.43	0.35	0.38	0.42	0.34	1.72	1.89	1.55	4.3	4.7	3.9	0.43	0.48	0.39
Mean Deviation divided by Mean..			0.47	0.46	0.46	0.55	0.56	0.56	0.36	0.35	0.37	0.37	0.34	0.37	0.43	0.41	0.44

change in these other factors. The first assumption implies a linear relation between strength of brick and wall strength. The second assumption modifies the first and introduces the conception of one variable modifying the action of another variable. The relations are likely to be anything except linear.

TABLE IX.—RATIOS OF COMPRESSIVE STRENGTH OF WALLETTES TO STRENGTHS OF BRICK.

Group	Brick	Number of Wallette Tests	Ratio, Wallette Strength to Compressive Strength of Brick Flatwise			Ratio, Wallette Strength to Compressive Strength of Brick on Edge			Ratio, Wallette Strength to Modulus of Rupture			Ratio, Wallette Strength to Tensile Strength of Brick			Ratio, Wallette Strength to Shearing Strength of Brick		
			Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
8 AND 12-IN. SOLID WALLETTES. MASON B. 1:$\frac{1}{10}$:3 CEMENT MORTAR																	
No. 1.	E H I	6 6 6	0.40 0.41 0.48	0.50 0.48 0.59	0.24 0.29 0.43	0.40 0.30 0.47	0.50 0.36 0.57	0.23 0.22 0.41	2.0 2.3 2.1	2.5 2.7 2.6	1.2 1.6 1.9	6.1 5.9 5.3	7.6 6.9 6.4	3.6 4.2 4.7	0.58 0.50 0.53	0.72 0.58 0.65	0.34 0.35 0.47
Mean Deviation divided by Mean..			0.08	0.08	0.22	0.15	0.17	0.29	0.04	0.03	0.12	0.05	0.06	0.10	0.06	0.08	0.15
8 AND 12-IN. SOLID WALLETTES. MASON A. 1:3 CEMENT MORTAR																	
No. 2.	A J	5 6	0.23 0.29	0.26 0.33	0.21 0.26	0.20 0.43	0.23 0.50	0.19 0.39	0.59 2.1	0.67 2.5	0.53 1.9	1.8 5.6	2.1 6.6	1.7 5.1	0.35 0.96	0.40 1.12	0.32 0.86
Mean Deviation divided by Mean..			0.12	0.12	0.11	0.36	0.37	0.34	0.56	0.58	0.56	0.51	0.52	0.50	0.47	0.47	0.46
8 AND 12-IN. SOLID WALLETTES. MASON B. 1:1:6 CEMENT-LIME MORTAR																	
No. 3.	E H I	6 6 6	0.31 0.28 0.38	0.40 0.33 0.51	0.21 0.20 0.29	0.31 0.21 0.37	0.39 0.25 0.50	0.21 0.15 0.28	1.5 1.5 1.7	2.0 1.8 2.2	1.05 1.1 1.3	4.7 4.0 4.2	6.0 4.7 5.6	3.2 2.9 3.2	0.44 0.34 0.42	0.56 0.40 0.56	0.30 0.24 0.32
Mean Deviation divided by Mean..			0.12	0.15	0.17	0.20	0.24	0.19	0.06	0.07	0.09	0.07	0.09	0.04	0.10	0.12	0.10
8 AND 12-IN. SOLID WALLETTES. MASON A. 1:1$\frac{1}{4}$:6 CEMENT-LIME MORTAR																	
No. 4.	A J	6 6	0.20 0.21	0.22 0.25	0.18 0.18	0.18 0.32	0.19 0.38	0.17 0.27	0.52 1.56	0.56 1.86	0.47 1.32	1.6 4.2	1.8 5.0	1.5 3.5	0.31 0.71	0.33 0.85	0.28 0.60
Mean Deviation divided by Mean..			0.02	0.06	0.00	0.28	0.33	0.23	0.50	0.54	0.48	0.45	0.47	0.40	0.39	0.44	0.36
8 AND 12-IN. SOLID WALLETTES. MASON A. 1$\frac{1}{4}$:3 LIME MORTAR																	
No. 5.	A J	6 6	0.12 0.10	0.12 0.11	0.10 0.087	0.10 0.15	0.11 0.17	0.09 0.13	0.30 0.75	0.32 0.84	0.27 0.64	0.93 2.00	1.00 2.2	0.84 1.7	0.18 0.34	0.19 0.38	0.16 0.29
Mean Deviation divided by Mean..			0.09	0.04	0.07	0.20	0.21	0.18	0.43	0.45	0.41	0.36	0.38	0.34	0.31	0.33	0.29

Take the first group of walls in Table VII built by Mason B with 1:3 cement mortar. For compressive strength flatwise, bricks *A*, *E*, and *H* give ratios in fair agreement. The wall strength associated with brick *I* is too high in comparison. Taking compressive strength on edge as the measure, not only does lot *I* remain high but lot *H* becomes low. For modulus of rupture bricks *E*, *H* and *I* are in excellent agreement. The ratio for brick *A* is less than half of those of the others. The ratios for wall strength to tensile strength resemble those for wall strength to modulus of rupture: fair agreement for bricks *E*, *H*, *I* and a low ratio for brick *A*. Ratios for shear are fair for all four bricks.

TABLE X.—RATIOS OF COMPRESSIVE STRENGTH OF WALLETTES AT FIRST CRACK TO STRENGTH OF BRICK.

Group	Brick	Number of Wallette Tests	Ratio, Wallette Strength to Compressive Strength of Brick Flatwise			Ratio, Wallette Strength to Compressive Strength of Brick on Edge			Ratio, Wallette Strength to Modulus of Rupture			Ratio, Wallette Strength to Tensile Strength of Brick			Ratio, Wallette Strength to Shearing Strength of Brick		
			Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
8 AND 12-IN. SOLID WALLETTES. MASON B. 1:$\frac{1}{10}$:3 CEMENT MORTAR																	
No. 1.	E	6	0.36	0.47	0.18	0.36	0.46	0.18	1.8	2.3	0.88	5.5	7.0	2.7	0.52	0.67	0.25
	H	6	0.25	0.36	0.14	0.18	0.27	0.11	1.4	2.0	0.80	3.6	5.1	2.1	0.30	0.43	0.17
	I	6	0.43	0.50	0.40	0.42	0.48	0.38	1.9	2.2	1.74	4.7	5.5	4.3	0.47	0.55	0.44
Mean Deviation divided by Mean..			0.17	0.14	0.42	0.28	0.23	0.45	0.12	0.05	0.35	0.15	0.14	0.27	0.21	0.15	0.36
8 AND 12-IN. SOLID WALLETTES. MASON A. 1:3 CEMENT MORTAR																	
No. 2	A	5	0.20	0.26	0.12	0.18	0.23	0.12	0.51	0.67	0.31	1.6	2.1	0.98	0.31	0.40	0.19
	J	6	0.22	0.24	0.19	0.33	0.36	0.29	1.6	1.8	1.4	4.3	4.8	3.8	0.73	0.81	0.65
Mean Deviation divided by Mean..			0.05	0.04	0.23	0.29	0.22	0.41	0.52	0.46	0.64	0.46	0.39	0.59	0.40	0.34	0.55
8 AND 12-IN. SOLID WALLETTES. MASON B. 1:1:6 CEMENT-LIME MORTAR																	
No. 3.	E	6	0.23	0.36	0.13	0.23	0.36	0.13	1.2	1.8	0.66	3.5	5.5	2.0	0.33	0.52	0.19
	H	6	0.18	0.22	0.12	0.13	0.16	0.09	0.98	1.2	0.67	2.5	3.1	1.7	0.21	0.26	0.15
	I	6	0.35	0.50	0.19	0.34	0.49	0.18	1.5	2.2	0.84	3.8	5.5	2.1	0.38	0.56	0.21
Mean Deviation divided by Mean..			0.24	0.25	0.20	0.30	0.38	0.23	0.15	0.23	0.11	0.15	0.23	0.09	0.21	0.27	0.13
8 AND 12-IN. SOLID WALLETTES. MASON A. 1:$1\frac{1}{4}$:6 CEMENT-LIME MORTAR																	
No. 4.	A	6	0.17	0.21	0.11	0.15	0.19	0.10	0.43	0.54	0.28	1.3	1.7	0.90	0.26	0.32	0.17
	J	6	0.15	0.17	0.13	0.23	0.25	0.20	1.13	1.25	0.98	3.0	3.3	2.6	0.51	0.57	00.45
Mean Deviation divided by Mean..			0.06	0.11	0.08	0.21	0.14	0.33	0.45	0.40	0.56	0.40	0.32	0.48	0.32	0.28	0.45
8 AND 12-IN. SOLID WALLETTES. MASON A. $1\frac{1}{4}$:3 LIME MORTAR																	
No. 5.	A	6	0.072	0.095	0.041	0.064	0.085	0.036	0.18	0.24	0.10	0.58	0.77	0.33	0.11	0.15	0.06
	J	6	0.062	0.072	0.060	0.094	0.109	0.091	0.46	0.54	0.45	1.23	1.43	1.19	0.21	0.24	0.20
Mean Deviation divided by Mean..			0.07	0.14	0.19	0.19	0.12	0.42	0.44	0.38	0.63	0.36	0.30	0.57	0.31	0.23	0.54

Passing to the group of walls built by Mason A also using cement mortar, fair agreement is found for ratios of wall strength to compressive strength for bricks tested flatwise. Brick *J* is out of line for the compressive strength edgewise. Brick *A* gives a low ratio for modulus of rupture and also for tension, while brick *J* gives rather high ratios for these two measures of strength. Brick *J* is also in poor agreement for the shear ratio.

It is unnecessary to go over the rest of this table in detail. Except for Group 3, which is restricted to soft-mud and dry-pressed brick, and Groups 6 and 7 likewise restricted, it is found that compressive strength flatwise gives the best tie-in of the various brick strengths. Modulus of rupture gives very excellent agreement for the three soft-mud and dry-pressed brick, but the end-cut brick *A* is in very poor agreement with the others on this basis. What has been said for modulus of rupture as a measure applies to tensile strength also. Shear is apparently not a very good measure, especially failing to rate properly the sand-lime brick *J*.

The two sections devoted to the 4-in. or "Economy" wall illustrate very strikingly a tendency also shown by the other solid constructions. The ratio of wall strength to brick strength for the strong brick *H* is reduced proportionately more than the ratio for brick *I* when a weaker mortar is used. Comparing the cement and cement-lime mortars, when brick *H* is used, the cement-lime-mortar walls gave 64 per cent of the strength of the cement-mortar walls. When brick *I* is used the corresponding value is 89 per cent. The same thing holds for the 8 and 12-in. walls.

The opinion has been expressed that it is not the maximum compressive strength of masonry but the strength at the first sign of weakness which is the important value. Table VIII gives ratios of strength at first crack to strengths of brick. The table deals with the same specimens, arranged in the same order as in Table VII. By "first crack" is meant the first visible evidence of incipient failure observed during the loading of the wall. Where lime mortar was used, the first evidence of failure was usually a perceptible crushing of the mortar joints and forcing of powdered mortar out of the joints. With stronger mortars, frequently the first sign of failure was breaking of the top header course. On continuing to apply load, other headers broke, but failure of bricks by spalling was always in evidence before the maximum load had been applied.

Space does not permit a discussion of the mechanics of masonry failure. However, it should be noted that the type of failure is influenced by the mortar and the workmanship as well as by the properties of the bricks themselves.

Table IV gives ratios for strength at first crack to maximum strength for all the walls here considered. It will be noted that walls built from bricks *A* and *I* show no signs of weakness until near maximum load, while walls built from bricks *E*, *H* and *J* show signs of impending failure relatively early. Reference to these facts will explain the variation in Table VIII as compared with Table VII. As a matter of speculation, the suggestion is made that brick *A* behaves in this manner on account of its higher modulus of rupture, while *I* depends upon regularity of shape.

Tables IX and X, comparing wallette strength with brick strength, give no new relations. The ratios of wallette strength at first crack and maximum to the various brick strengths merely confirm the observations made for Tables VII and VIII.

Table XI.—Ratios of Wall Strength to Compressive Strength of Brick According to Distribution.

Walls built by Mason B. 1:$\frac{1}{10}$:3 cement mortar

	Brick A		Brick E		Brick H		Brick I	
	Brick Compressive Strength, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Compressive Strength, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Compressive Strength, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Compressive Strength, lb. per sq. in.	Ratio, Wall Strength to Brick Strength
Average Strength of Walls, lb. per sq. in.	954		1145		2710		1510	
Brick strength								
100 per cent less than	5460	0.175	4800	0.238	12 250	0.221	7660	0.197
90 per cent less than	4040	0.236	4050	0.283	10 630	0.255	5100	0.296
80 per cent less than	3770	0.253	3740	0.306	10 000	0.271	4380	0.345
70 per cent less than	3550	0.269	3520	0.325	9 300	0.290	3650	0.414
60 per cent less than	3440	0.277	3460	0.331	9 030	0.300	3330	0.453
50 per cent less than	3330	0.286	3240	0.354	8 710	0.311	3150	0.480
40 per cent less than	3190	0.299	3050	0.375	8 000	0.339	3040	0.497
30 per cent less than	3060	0.312	2830	0.405	7 470	0.363	2900	0.521
20 per cent less than	2880	0.331	2740	0.418	6 980	0.389	2760	0.547
10 per cent less than	2670	0.357	2660	0.430	6 540	0.414	2570	0.588
5 per cent less than	2400	0.398	2525	0.453	5 570	0.487	2450	0.616
0 per cent less than	2140	0.445	2110	0.543	3 900	0.695	1850	0.816
Mode	3250	0.30	2750	0.42	9 250	0.29	3000	0.50
Mean	3370	0.28	3325	0.34	8 610	0.31	3515	0.43

There remains only to consider the possibility of the strength of masonry having a relation to the distribution of the brick strengths. Hitherto averages have been reported and discussed. What is now proposed is to investigate the theory that the strength of masonry is determined by the strength of the weaker brick in the wall.

Table XI was constructed in the following manner: The results from all of the compressive strength flatwise tests for a given make of brick were arranged in order of magnitude, starting with the highest value and ending with the lowest. Taking the column for brick *A*, the value 5460 lb. per sq. in. is the strength of the strongest brick tested. The figure 4040 represents the strength equalled or exceeded in 10 per cent of the tests, the results from the remaining 90 per cent being lower than this value. In like manner values were selected down to 2140 lb. per sq. in., which is the strength of the weakest brick. Ratios of wall strength to these partial brick strengths were calculated and are presented in the right-hand column.

Unless the absurd conclusion is accepted that the strength of the wall is controlled by the strength of the single strongest brick used, the conclusion is justified from Table XI that no improvement in constancy of ratios is introduced by using any other measure than the mean of the compressive strength measured flatwise.

TABLE XII.—RATIOS OF WALL STRENGTH TO MODULUS OF RUPTURE OF BRICK ACCORDING TO DISTRIBUTION.

Walls built by Mason B. 1 : $\frac{1}{10}$: 3 cement mortar

	Brick A		Brick E		Brick H		Brick I	
	Brick Modulus of Rupture, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Modulus of Rupture, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Modulus of Rupture, lb. per sq. in.	Ratio, Wall Strength to Brick Strength	Brick Modulus of Rupture, lb. per sq. in.	Ratio, Wall Strength to Brick Strength
Average Strength of Walls, lb. per sq. in.	954		1145		2710		1510	
Brick strength								
100 per cent less than	2530	0.38	1150	1.00	2740	0.99	1590	0.95
90 per cent less than	2050	0.47	845	1.36	2020	1.34	1090	1.38
80 per cent less than	1800	0.53	780	1.47	1890	1.43	975	1.55
70 per cent less than	1660	0.57	760	1.50	1710	1.58	885	1.71
60 per cent less than	1465	0.65	722	1.59	1620	1.67	790	1.91
50 per cent less than	1250	0.76	663	1.73	1545	1.75	765	1.97
40 per cent less than	1120	0.85	635	1.80	1410	1.92	720	2.10
30 per cent less than	1040	0.92	590	1.94	1345	2.01	680	2.22
20 per cent less than	840	1.14	535	2.14	1260	2.15	630	2.40
10 per cent less than	650	1.47	450	2.54	1085	2.50	555	2.72
5 per cent less than	530	1.80	405	2.83	980	2.76	450	3.35
0 per cent less than	405	2.35	151	7.59	853	3.18	301	5.01
Mode	1300	0.72	650	1.76	1475	1.83	750	2.02
Mean	1318	0.72	671	1.71	1550	1.75	798	1.89

Table XII is similarly constructed for the same bricks and walls to show effect of considering distribution of the modulus of rupture. It will be noted that the low ratio characterizing the modulus of rupture for brick *A* does not change greatly relative to those for the other bricks throughout the distribution. However, in justice to distribution as a factor in wall strength it should be noted that while Tables II and III show considerable variation in distribution, unfortunately other properties of the brick also vary widely. Hence, it must be admitted that these data are not decisive.

Before giving a formal summary, a few points should be clarified. By the statement that the compressive strength of brick tested flatwise is the most consistent measure of wall strength, workmanship, mortar and type of construction being equal, is not meant that wall strength is proportional to brick strength. What is meant is that if two lots of bricks have the same compressive strength tested flatwise and the mortar, workmanship, and type of construction are the same, the same wall strength will probably result. Variation from this relationship in the present series of tests is readily explainable by application of the unfortunately qualitative factor of regularity of size and shape of brick unit. Or putting the same idea in different words, the statement means that if two lots of brick have the same compressive strength flatwise and are equally uniform and regular in shape and size but differ in modulus of rupture, when workmanship, mortar, and type of construction are the same, equal compressive strength of masonry will result. Conversely, if the modulus of rupture is the same for both sets of brick and compressive strength is different, the wall strengths will be different. The above statement is offered simply as a hypothesis suggested by these tests and is subject to confirmation or refutation by new data.

The reader will have noticed long before this that the ratios, wall strength to brick strength, especially when "brick strength" means compressive strength, are the same as what are called by some writers "efficiencies." It is hardly necessary to point out that these "efficiencies" are very much controlled by factors outside of properties of the bricks themselves; hence comparing bricks on the basis of these ratios, where workmanship, strength of mortar, etc., vary, is likely to lead to some rather misleading conclusions.

CONCLUSIONS

Considering the paper as a whole the following points are deemed of sufficient importance to warrant inclusion in the summary:

1. The various measures of the strength of brick, compressive, flatwise and on edge, transverse, tensile and shearing, vary in their relation one to another for different makes of brick.

2. The factors, workmanship, strength of mortar, types of construction and regularity of size and shape of brick being equal, compressive strength of brick, flatwise, is the most consistent measure of compressive strength of brick masonry on the basis of the data here presented.

3. For the soft-mud and dry-pressed brick the ratios of compressive strength flatwise to transverse strength are fairly constant. The transverse strength of these bricks is as good a measure of wall strength as the compressive flatwise strength—possibly better.

4. The ratio, wall strength to brick strength, varies with variation in workmanship, strength of mortar, type of construction and regularity of size and shape of brick.

5. Where there is a difference in strength between bricks, the ratio of wall strength to brick strength for the stronger brick is more affected by change of mortar strength than is the corresponding ratio for the weaker brick.

6. All other factors being equal, the brick most regular in size and with plane, parallel faces, will give the highest masonry strength.

7. The ratio, strength at first crack to maximum strength of masonry, seems influenced especially by the ratio of transverse to compressive strength and the regularity in size and shape of the bricks.

8. On the basis of these data, the mean of a number of determinations of brick strength is as good a measure of brick strength, in so far as it affects masonry strength, as either the norm or mode.

Acknowledgments:

The author acknowledges his indebtedness to Mr. A. H. Stang and Mr. D. E. Parsons with whom he has been associated in this work at the Bureau of Standards. Mr. H. V. Johnson very kindly supplied hitherto unpublished data which throw considerable light on some of the matters discussed here. Much assistance was received through the cooperation of the Division of Clay and Silicate Products and the Engineering Mechanics Section.

DISCUSSION

Mr. Palmer. MR. L. A. PALMER[1] (*presented in written form*).—In Mr. McBurney's very excellent paper, there is no mention of ordinary shale, stiff-mud, side-cut brick. Manufacturers of this type of brick report that tests have shown that such brick usually have a compressive strength flatwise exceeding 12,000 lb. per sq. in. There is a considerable quantity of this type of common brick being manufactured, and approximately 90 to 95 per cent of face brick are of this type. Any future work should certainly include tests of this type of brick.

Mr. Ingberg. MR. S. H. INGBERG[1] (*presented in written form*).—The author presents results of strength tests of individual bricks and of masonry in greater volume and detail than has heretofore been done. The care with which representative samples of individual bricks were obtained indicates that for the bricks included in the tests such relations as are obtainable between strength of the units and the strength of the masonry have been fairly well defined. The limitations, so far as conclusions of general application are concerned, would be due to the relatively small number of sources represented as compared with the wide range in properties and relation of properties the field presents.

Thus, as it concerns the relation of modulus of rupture and compressive strength flatwise of clay bricks, there are shown in the accompanying Fig. 1 compressive strength and modulus of rupture, both obtained with bricks tested flatwise, of about 150 lots of clay and shale bricks. While this includes data collected from several sources in connection with the formulation of specifications for brick by the Federal Specification Board, most of the data are from tests conducted under the auspices of the Society's Committee C-3 on Brick.[3] The values for the four clay bricks, bricks *A*, *E*, *H* and *I*, included in the author's comparison, are also given. It is seen that the ratio of modulus of rupture to compressive strength flatwise ranges all the way from one-sixteenth to nearly one-half as based on the average results from five or more tests. The ratios of other properties have proportionate ranges. This variation in properties and their ratio to each other would indicate possibilities of serious limitations on general conclusions as to relationship of masonry strength and brick strength derived from tests of bricks from a relatively few sources.

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[1] Research Associate, American Face Brick Association, U. S. Bureau of Standards, Washington, D. C.

[2] Senior Engineer, U. S. Bureau of Standards, Washington, D. C.

[3] *Proceedings*, Am. Soc. Testing Mats., Vol. XV, Part I, p. 150 (1915); Vol. XIX, Part I, p. 266 (1919).

Mr. Ingberg.

The ratios of compressive strength of walls and wallettes to the various types of brick strength given in Tables VII to X indicate large deviations for the bricks included—so large that even for those pertaining to the compressive strength flatwise, adjudged by the writer to be the best measure of masonry strength, the deviations are of such amount that as a measure of masonry strength it cannot be regarded as consistent and satisfactory. It is apparent that for groups containing three or more results that are compared for consistency, the maximum deviation divided by the mean value will generally be larger than the author's criterion, "mean deviation divided by mean," being for some values in groups Nos. 1 and 2 of his Tables VII and VIII as much as 100 per cent higher. At the 1924 meeting of this Society, I presented a paper[1] giving the

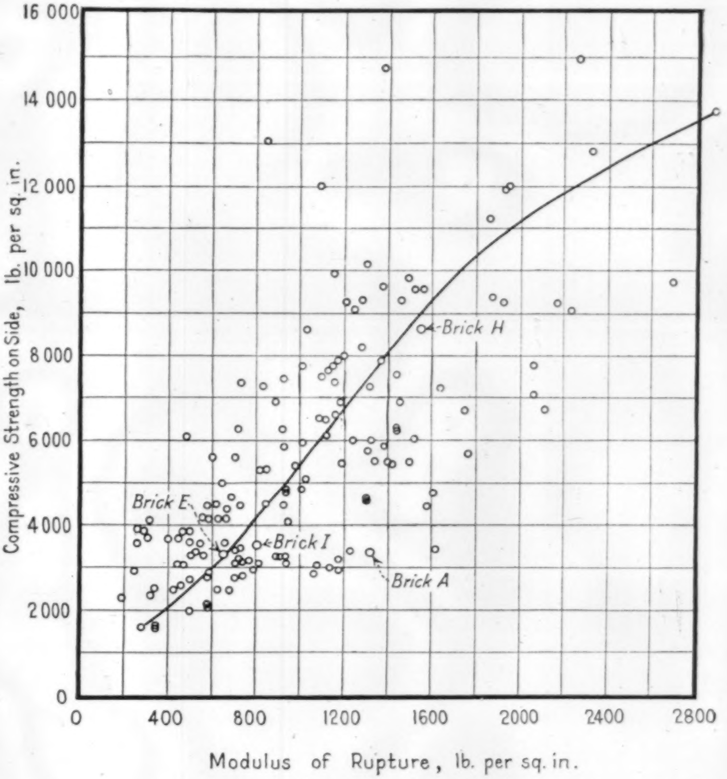

FIG. 1.—Compressive Strength on Side and Modulus of Rupture of Clay and Shale Bricks.

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[1] "Factors Affecting Brick Masonry Strength," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 909 (1924).

Mr. Ingberg. results of strength tests of masonry specimens constructed in connection with fire tests of brick walls, these being correlated with the strength of individual bricks constituting the masonry. Here also large deviations were found in the ratio of masonry strength to brick strength as it concerned the different kinds of bricks included. While, as stated in my paper, "It is apparent from the results that there is a general increase in the strength of masonry with increase in the strength of individual bricks, this applying for compressive strength of bricks tested flatwise, on edge and for the transverse strength," this is nothing but a general trend. In the given paper striking exceptions are indicated, the results for sand-lime bricks from one source and for concrete bricks from one source coming outside of the general trend for bricks from four other sources for which the general statement holds.

This lack of a consistent relation between strength of individual bricks and masonry strength would indicate the desirability of beginning a search for a more reliable measure of masonry strength that can be conveniently applied. That we have for brick masonry no control test as reliable as the cylinder test for concrete is also apparent. A test that suggests itself would be of four or five bricks bedded one above the other in the same kind of mortar as that for the masonry concerned. This specimen should give opportunity for approximately the same combination of resisting forces as in masonry with bricks laid flatwise. That the compressive test flatwise of single bricks does not give this same condition is very apparent, the brick being restrained by the bearings.

That the character of the surface of the brick units and the degree of bond effected with the mortar has no relation to masonry strength cannot be taken without reservation so far as application to a wide range in brick and mortar is concerned. The tests quoted were made by pulling apart bricks bonded with cement mortar, the bricks having previously been boiled five hours in the absorption test. The surface of some bricks is covered with deposits that have been found to be mainly calcium or alkali sulfates formed during the drying or burning from lime or other salts initially present in the clay or applied to the raw molded bricks to prevent sticking. Other bricks have a gritty surface from the sand used for surfacing the molds. Sand-lime and concrete bricks also present a large range in surface characteristics, caused by differences in constituent materials and methods and pressures used in molding. The boiling treatment accorded the bricks before being used for the adhesion tests necessarily changed the surface characteristics

Mr. Ingberg.

of the bricks. Also; the method of testing does not represent the way in which the bond is stressed in the wall, the direction of the forces of which are probably more nearly parallel with than perpendicular to the bedding planes. Since it must be admitted that a masonry pier built of the standard brick units is weaker than a similar monolithic pier of the same material, the difference in strength is dependent on the properties of the mortar and the adhesion with the brick units, and it would appear that the more nearly a monolithic body is obtained by means of the mortar and its adhesion to the units the higher will be the masonry strength.

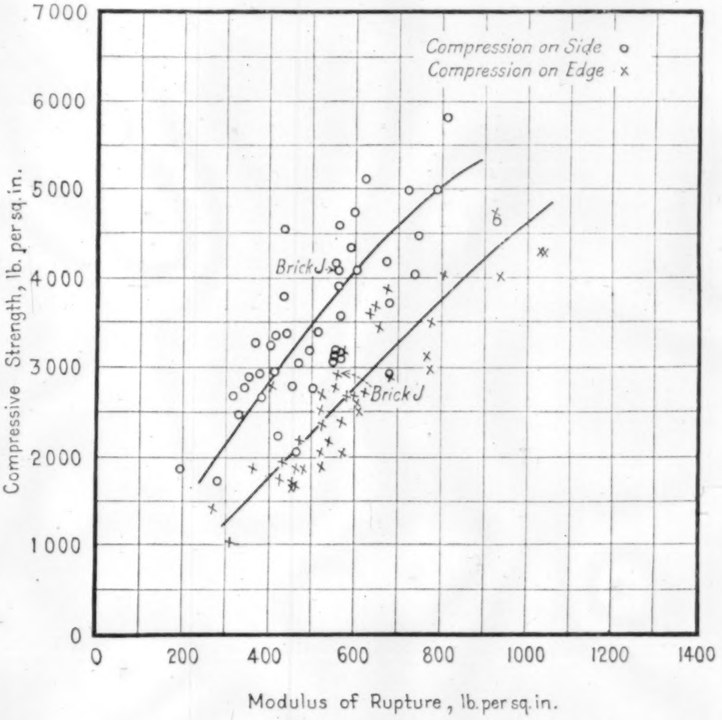

FIG. 2.—Compressive Strength on Side and Edge and Modulus of Rupture of Sand-lime Bricks.

Another point to which exception can be taken is the rejection of the transverse or tensile strength of individual bricks as giving any information of value on the strength of the masonry. For bricks that are low in tensile or transverse strength as compared with the compressive strength it cannot be premised that the former has no relation to the strength of the masonry. As previ-

Mr. Ingberg. ously indicated, the compression tests of individual bricks does not bring into action the internal resisting forces in the same manner as in the masonry specimen. The author indicates that the transverse and tensile strength of individual bricks may have some relation to the strength of masonry at first crack or the point where the headers begin to fail. From the standpoint of structural safety it is a question whether the ultimate strength of the wall is of greater importance than the strength at the point where the brick wythes begin to separate due to failure of header bond. At any rate, the data presented cannot be considered as conclusive since by omission of the results from the tests with brick *A* the author's conclusion relative to relation of transverse brick strength to masonry strength would have to be greatly modified. Sweeping conclusions cannot safely be drawn if they are based on an accidental circumstance such as the inclusion of bricks from this source in the present series. Brick *A* has very high transverse strength as compared with its compressive strength and it should be expected that the masonry strength would be limited by the latter. The test that would give information on whether the transverse strength of brick limits the strength of the masonry built with it would be with a brick having low transverse strength as compared with its com-

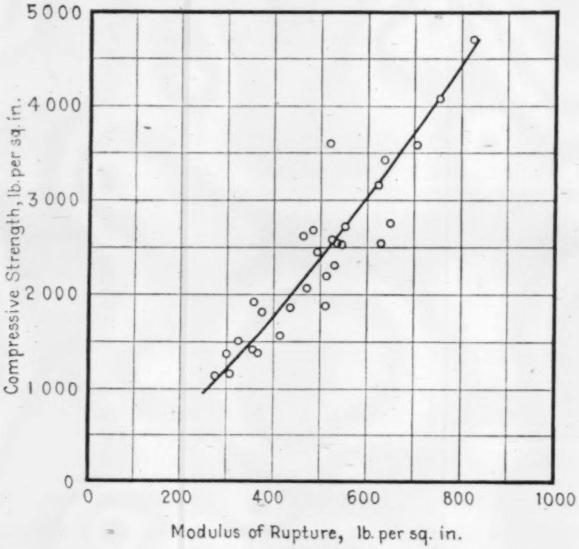

FIG. 3.—Compressive Strength on Side and Modulus of Rupture of Concrete Bricks.

Mr. Ingberg. pressive strength. It is significant, although not necessarily conclusive, that according to the author's data in groups comprising bricks *E*, *H* and *I*, whose properties fall near the general line of trend of the brick data shown in Fig. 1, the transverse strength of individual bricks is as consistent a measure of masonry strength as the compressive strength.

The scattering of points in Fig. 1 shows that generally for clay and shale bricks the compressive and the transverse strength cannot both be consistent measures of masonry strength as it pertains to all the sources represented in the diagram, and it must be left for further tests to determine more definitely the extent to which the one is a better measure than the other.

As it concerns sand-lime and concrete bricks, the relation between the compressive and the transverse strength is more consistent to judge from Figs. 2[1] and 3,[2] which show all available data published and unpublished relating to these properties for the products made in this country. It would appear, therefore, that for these types of brick, any relation that can be predicated to obtain between either of these properties and the masonry strength, can be expressed in terms of the other with as much precision as would be expected to hold for the relation between any property of the individual bricks and the masonry strength.

Mr. Lawson. MR. T. R. LAWSON.[3]—This paper is a very remarkable one in many respects. We are greatly indebted to Mr. McBurney for the painstaking care with which he has conducted this series of experiments. Further than this, it represents one of the most complete documents of its kind. His bibliography is particularly complete, and the paper will undoubtedly be one of great value to engineers and architects in the future. It is to be hoped that it is only the beginning, and that he has started something which either he or someone else will carry out along a somewhat broader field. For instance, we see no mention in this paper of concrete brick or of some of the various shale brick. It is highly desirable where an experimenter is working with a product that he include all of the materials that are available, in order to make the treatment general.

Mr. McBurney is to be congratulated on his attempt to eliminate many of the existing variables as he has tried to bring the

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[1] From tests conducted at the Bureau of Standards including unpublished data from H. V. Johnson's tests; also from other scattered sources and the literature: namely, Woolson, *Proceedings*, Sand-Lime Brick Assn. (1905); Lazell, *Proceedings*, Sand-Lime Brick Assn. (1908).

[2] Unpublished data from several sources collected in connection with work on specifications for brick by Federal Specifications Board, also from *Bulletin No. 2*, Department of Civil Engineering, Columbia University, New York City.

[3] Head, Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N. Y.

Mr. Lawson. question down to the variable of a single brick. It must be apparent to those who have read this paper carefully that we are at the point where a single kind of test is not necessarily indicative of the quality of the masonry itself. For instance, in respect to brick *H*, there was unusual strength in compression and very great strength in the brick wall. On the other hand, another type of brick from the middle section of the country is quite deficient in transverse strength, due largely to lamination, and yet it showed up well in the wall.

The method of manufacture, as to whether it is a stiff-mud wire-cut brick or whether it is a soft clay brick will therefore be a factor for the engineer to consider, for he must recognize that the brick from one section of the country will be very different in quality from those obtained from another section of the same country. It will not be easy to write a specification for brick which will include all of the bricks in this country, and emphasis should be made on the point that it will be necessary for engineers and architects in one section to know the peculiarities of their particular brick and emphasize them in their specifications.

There are just one or two little items in the paper which should be cleared up. On page 613, for instance, mention is made of the strength of the mortar. No reference is given to the age of this mortar. While it may not be of very great importance, yet it would be well to add that information.

Again, in the present compression tests of brick, as carried out at the Bureau, the surfaces were cushioned by plaster, and suggestion has been made that possibly it is a test of plaster rather than brick. Would it not, therefore, be well to include, if possible, a series of compression tests on similar brick or brick taken from the same lot, if they still exist, with the surfaces ground and the brick tested between flat plates? The question of strength of mortar would certainly be eliminated by doing this.

In a test of a wall the mortar will probably be found to be the weakest part; the kind of mortar that is used is therefore going to be indicative of the strength of the wall. Another test which Mr. Ingberg has also suggested comes to mind—that of making compression tests of brick that would be bonded together with mortar as in a wall. The tests of wallettes is an attempt to cover this same point. The wallette is of such a size that it can be tested in the average laboratory, and it is to be hoped that more data will be secured from testing wallettes. Obviously, it is impossible to build large walls and wait for them to age and test them in the ordinary

Mr. Lawson. commercial testing laboratory, but if more data could be obtained on the wallette itself and its relation to strength of wall they would be of very great interest and value.

Mr. McBurney. MR. J. W. MCBURNEY.[1]—Just a few words on the points that Mr. Lawson raised.

The mortar cylinders were tested at the same time as the walls. This gave an age of sixty days, plus or minus two days. The procedure was to cast the cylinders from a batch or batches of mortar just as it was being used in the walls. In some cases the material was taken from the mortar box, in other cases from the mason's board. The mortar was then permitted to set 24 hours in the mold, then capped with neat cement. After another 24 hours, the cylinders were removed from their molds and one-half were stored in water and one-half set on top of the walls. The water-cured cylinders were tested wet.

The question of the apparent strength of the individual brick as affected by the type of capping is, of course, of the greatest importance. My own experience has been that one can vary within rather larger limits the apparent compressive strength of brick by the type of capping used, such as, gypsum, neat portland cement, high-alumina cement, or by the use of a compound such as a mixture of clay and sulfur. The last, I believe, is recommended by the Pittsburgh Testing Laboratory. On an average, the highest results are obtained by the method which Mr. Lawson referred to, grinding smoothly the face of the brick. It seems to me that this is another argument for a standard method of testing bricks. The true strength is always more or less an open question, but if we all make our tests in the same way I think results would be comparable.

On the question of including other makes of brick, raised by Mr. Palmer and Mr. Lawson, it should be understood that this paper is not a comparison of different makes of bricks. It would have certainly added to the paper if several kinds of concrete bricks, shale bricks and adobe bricks had been used in masonry tests with the same workmanship, etc. With those data at hand, it would have been possible to seek and draw conclusions in a much wider field than was done. However, as mentioned before, this paper is a by-product from another investigation. The bricks were picked in this original investigation not for the purpose of covering the field of methods of manufacture, but for the sole purpose of representing each of the four grades of the Society's former Specifications for Building Brick. That the

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[1] Research Associate, Common Brick Manufacturers' Association of America, U. S. Bureau of Standards, Washington, D. C.

Mr. McBurney. bricks selected badly missed some of these grades is unfortunate for the original investigation, but the variety of ratios for strength, to my mind, warranted the present paper.

(*Author's closure by letter*).—Mr. Ingberg's criticism would be valid if the paper had concluded that the flat compressive strength of brick measured the strength of brick masonry without regard to variation in other factors. The comparison of the ratio of flat compressive strength of brick to compressive strength of brick masonry and the ratio of compressive strength of a concrete cylinder to compressive strength of concrete masonry seems to the author beside the point.

A concrete cylinder is a sample of the finished mass. The factors, water-cement ratio, mix, grading of aggregate, etc., are all included. It would seem that the wallette is the analogue of the cylinder. The proper comparison for the strength of the brick is the strength of neat cement or the cement - standard sand mortar. In short the paper attempted not to supply a measure of masonry strength, but to evaluate one of the several factors upon which the design of masonry to secure a definite strength must depend.

It is admitted that the paper is far from complete in that it has not covered all of the variations in ratios of strength of bricks. In particular no masonry tests were made on bricks characterized by very low ratios of transverse to compressive strength. The author has recently examined a very laminated side-cut brick which characteristically and consistently gave a ratio for modulus of rupture to flat compressive strength of 0.08. It is the intention to construct and test some walls of this brick with workmanship and other factors so that the results will be comparable with the results here reported.

TESTS OF CLAY AND CONCRETE LOAD-BEARING PIPE

By W. J. Schlick[1]

Synopsis

Our increasing knowledge of the conditions pipe must meet in service has made more general the testing of all sizes and classes of pipe. Although the test methods in the standards, and tentative standards, for drain tile, for sewer pipe and for reinforced concrete pipe are very similar, they are not identical in all their provisions. This paper discusses some of the problems of test procedure that are more commonly encountered.

The problem of securing a more general recognition of the need for testing and of the use of test results as a basis for the structural design of pipe lines is probably yet the most important one. The problems of such design have not all been solved, but the available information is sufficient, if properly used, to prevent most of the costly pipe failures.

Because of the wide use of the strength test, particular attention is given the problems encountered in the use of the two most common of the standard bearings, sand bearings and three-edge bearings, in tests of different sizes and classes of pipe and to the necessity for controlling the moisture and temperature of the specimens if truly comparable test results are to be obtained.

The real meaning and value of the absorption test result are subject to so much discussion and the prescribed methods so frequently are criticized as being unfair, that these questions are discussed. Certain common, but incorrect, procedures in the drying and immersion treatments are pointed out.

The freezing and thawing test is the only direct durability test that has been standardized. The development of this test, and experience in its use by one large consumer, are given briefly. The problems of durability against chemical attack have been solved only partially; discussion of them is deferred till more definite and conclusive research data are available.

The structural failures of pipe in service twenty years ago were being explained on almost every basis, though but seldom was it believed that the failure could have resulted simply because the pipe were overloaded. In 1911, the Society's pipe committee, Committee C-6 on Drain Tile, was organized. In 1913, Dean Anson Marston and A. O. Anderson of the Iowa State College published the results of their study of the loads on pipes in ditches. From this beginning the advance has been rapid; many of the loads and conditions which the pipe must meet in service have been determined and have been accorded ever-increasing recognition. There are now available data and formulas for determining the probable loads upon pipe under almost all conditions of service, data relating to special cases where durability is a major factor and standard quality and test specifications covering clay and concrete load-bearing pipe for use under most of the service conditions. Some of the problems of the design of such pipe lines to insure safety have not been solved, but the existing information, if properly used, is sufficient to prevent most of the costly pipe failures. The wider use of test methods, and their extension to other qualities, sizes and classes of pipe, have brought out many problems of test procedure. This paper discusses some of the problems suggested by recent inquiries and discussions.

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[1] Drainage Engineer, Engineering Experiment Station, Iowa State College, Ames, Iowa.

THE NEED FOR TESTING

The old problem of the necessity for testing pipe, or the real economy of such a practice, probably still is the most important one. Standard quality and test specifications, as those for drain tile[1] and sewer pipe[2] of this Society, those for reinforced concrete culvert pipe of the Joint Concrete Culvert Pipe Committee[3] and those for reinforced concrete sewer pipe of the American Concrete Institute, have done much to bring the idea of purchasing and testing pipe under such specifications to the attention of engineers, public officials and pipe manufacturers, and have furnished detailed descriptions of satisfactory test methods. In spite of the advance that has been made, there are still some engineers who think that they can look at a pipe, purchased under such general requirements as "No. 1" or "first class," and determine that it will be satisfactory, without giving any special consideration to the loads it may receive or to the agencies whose attacks may shorten its useful life. Likewise, there are some manufacturers who "just know" that their pipe are satisfactory for any installation where pipe may be used. Most of the costly pipe failures are the results of these attitudes and could have been prevented through the use of information readily available. The testing of pipe necessitates expenditures for equipment, for pipe and for labor, but if intelligent pipe testing is uneconomical so too would be the testing of other materials and products. If the original cost of most pipe lines where failures have occurred had been increased sufficiently to cover testing and more careful design, repairs amounting to 50 to 100 per cent of the original cost would have been unnecessary.

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[1] Standard Specifications for Drain Tile (C 4 - 24), 1927 Book of A.S.T.M. Standards, Part II. p. 190.

[2] Standard Specifications for Clay Sewer Pipe (C 13 - 24) and Standard Specifications for Cement-Concrete Sewer Pipe (C 14 - 24), 1927 Book of A.S.T.M. Standards, Part II, pp. 168, 179.

[3] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 319 (1928).

Testing should have a very definite place in every pipe manufacturer's operations. If he makes tests of his pipe at frequent intervals, he knows definitely whether he is maintaining the standard of his product. His test records furnish a logical basis for considering his bid on any job where the conditions are such that ordinary standard pipe probably will be unsafe. Even though the manufacturer furnishes pipe which meet the requirements of the job specifications, he and every other manufacturer of a similar class of pipe will suffer because of a failure in the constructed pipe line.

THE FUNCTION OF STANDARD SPECIFICATIONS

The test methods of the standard pipe specifications frequently are criticized because they do not duplicate the conditions under which the pipe will be used. The standard specifications prescribe by means of limits, the qualities or properties which the pipe must have to be satisfactory under specified general conditions of service and test methods by which compliance with these requirements may be determined under conditions that will be uniform in all tests made in accordance with them. So long as the relationships between the qualities shown by the standard tests and those required of the pipe in service are known, the tests will furnish the necessary information. The conditions of service vary so greatly that at best only a few of them could be approximated by the test methods of a standard specification. Although strength tests with sand bearings do show directly the supporting strength which drain tile and sewer pipe will develop under one of the ordinary conditions of service, each of the standard test bearings was suggested as a means of determining the strength of the pipe on a truly comparable quality basis. This test result may be used as a basis for determining, at least approximately, the supporting strengths which similar pipe will develop under the probable conditions in any installation. In the standard absorption test neither the drying nor the immersion treatment duplicates any condition normally encountered in service. However, the percentage of absorption determined in a comparatively short time in the standard test is no higher than that which is probable after the pipe is installed.

CARELESSNESS IN MAKING TESTS

The standard pipe tests are not of great refinement, and often are made carelessly because of a feeling that the test is so simple that there is no need for special study of the prescribed method or close adherence to it. The test methods are so fully described in the standard specifications, that a little careful study of the standard covering the class of pipe to be tested, and the test to be made, will prevent most of the controversies over test results. As now prescribed the tests for drain tile, sewer pipe, and reinforced-concrete culvert pipe are very similar, but there are some differences which must be observed if the tests are to be in compliances with the standard to be used.

STRENGTH TESTS MADE WITHOUT A TESTING MACHINE

The record of a strength test made by piling weight directly on the upper bearing has a special appeal to the layman, particularly when accompanied by photographs showing the great pile of pig iron or of sacks of sand or cement. Although most such tests are made for their spectacular value rather than to secure information as to the real quality of the pipe, an accurate test can be made in this way, and may sometimes be necessary if there is no testing machine available.

This method of loading, as a strength test method, should be limited to tests with sand bearings. The use of other beddings and bearings may give test results that can not be correlated with the requirements of any of the standard specifications. To secure a fair test, the load must be placed with considerable care to insure that it is in compliance with the conditions specified.

BEARINGS TO BE USED IN MAKING STRENGTH TESTS

Although each of the common pipe-test bearings (sand bearings, two-edge bearings and three-edge bearings) have their particular advantages and disadvantages, the three-edge bearings are so much more convenient to use that they have largely displaced the others for all routine tests. The results of tests of many lots of clay and concrete drain tile and sewer pipe show that the three bearings give equally reliable and uniform results, and that the average value of the ratio between results with sand bearings and with two-edge or three-edge bearings is near 1.50.[1] The fact that this is an average value indicates the probability that the comparative results for a single lot of pipe will not show just this value. Although the value of the ratio has not been checked extensively through tests of large pipe, the results available indicate the correctness of this value for the ratio. The use of three-edge bearings for tests of large hub-and-spigot pipe presents a special problem in this connection.

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[1] This ratio is given as 1.5 in the Society's specifications for drain tile and as 10/7 in its standards for sewer pipe.

MOISTURE CONDITION AND TEMPERATURE OF STRENGTH-TEST SPECIMENS

The controlling of moisture and of temperature are so troublesome, and their effects so unappreciated, that in many instances they are ignored. However, the importance of these controls, and the differences in the specification requirements, warrant special consideration.

Moisture Condition:

The available data indicate that the moisture condition of the pipe must be uniform if truly comparable data are to be obtained by the strength test. This is true of some of the softer-burned clay tile and is particularly important in testing concrete pipe. Such data as are available show that wetting concrete reduces the strength from 5 to 35 per cent, though they do not show the effects of different percentages of moisture or the qualities of the concrete that cause the reduction in strength to vary between these wide limits. The very fact that these effects are possible and are unexplained makes it necessary that the moisture condition of the test specimens be controlled carefully.

The provision regarding wetting in the Society's specifications for drain tile intends that the tile shall be wet when tested; the provisions in the other standard specifications intend that the pipe shall be nearly dry, or "air dry," when tested. The importance of each of these provisions is sufficient to warrant any extra care that compliance with it may require. These differences in method should be considered in comparing the results of tests made under the different specifications. The provision in the drain tile specifications should be made more definite and more applicable to the larger sizes of tile. The term "visible moisture" in the sewer pipe specifications should be defined so as to show definitely whether it refers to moisture that is visible, or to a visible effect of the presence of moisture.

Temperature Control:

Both the drain tile standards and the Joint Committee specifications for reinforced-concrete culvert pipe prohibit the testing of pipe when they are frozen or are at a temperature below 40° F. The sewer pipe standards do not cover this important point. In the early work of Committee C-6 it was found that some drain tile were much stronger when they were frozen. Some data secured by the author in connection with another study and now being prepared for publication, indicate that temperature control, at least for concrete pipe, may be just as important as moisture control. If a fair test is to be secured certainly no pipe should be tested when it is frozen or when it is at a temperature below 40° F.

STRENGTH TESTS WITH SAND BEARINGS

There are three factors in connection with strength tests using sand bearings which too often are ignored.

Condition of the Sand in the Bearings:

As originally adopted the Society's specifications for strength tests, with sand bearings, of both drain tile and sewer pipe provided simply that the sand used in the bearings should be clean and should pass a No. 4 sieve. With the intent of insuring more uniform test results, the drain tile specifications were later amended to provide that this sand should also be "dry." Since the author was one of those who suggested this change, he feels free now to criticize it. In 1926 Mr. D. G. Miller[1] presented the results of a study which showed that the distribution of pressure is more nearly uniform and the test results more uniform when moist sand is used and when the sand in the lower bearing is loose when the pipe is placed. These precautions should be taken. It will always be possible to obtain moist sand; obtaining dry sand is sometimes very inconvenient. The data available do not indicate that the use of moist sand will change the established ratios between the results with sand bearings and two or three-edge bearings.

Contact Between the Top Bearing Frame and the Pipe:

The specifications for strength tests using sand bearings all provide that the top bearing frame shall not be allowed to come in contact with the pipe nor with the top bearing plate. This provision is definite, but a convenient procedure that complies with it often is not known, with the result that the provision is more or less ignored. It may be complied with by placing the frame directly on the pipe for filling, and then raising it as soon as the load is sufficient to produce a side pressure that will support the frame. If the upper bearing plate is made slightly smaller than the inside of the frame, and is correctly placed, the prevention of contact between it and the frame presents no particular difficulty. However, if these two points are not watched, load will be transmitted through the sides of the bearing frame directly to the pipe, producing an application of load materially different from that intended.

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[1] D. G. Miller, "Determination of Pressure Distribution on Circular Pipe when Tested in the A.S.T.M. Standard Sand Bearings," *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part II, p. 611 (1926).

Rigidity of Top Bearing Plate and the Bearing Frames:

The requirement that the top bearing plate and the bearing frames shall be so rigid that they will not bend appreciably during the loading presents no small problem when testing large pipe. The frame of the lower bearing is built in place and is not handled, so that it may be made as heavy as necessary. The load may be applied to the top bearing plate through supplemental members so as to provide the required rigidity and distribution. The principal difficulty is with the top bearing frame. When testing large pipe, the sides of this frame are so long and the side pressure in the sand so high that a frame of the stiffness specified is so heavy that handling it is very difficult. A strict compliance with this provision when testing large pipe in long lengths will require a frame that is comparatively very heavy and very costly.

STRENGTH TESTS WITH THREE-EDGE BEARINGS

There are three problems, in making strength tests with three-edge bearings, which deserve special attention. These will be discussed briefly.

Tests of Hub-and-Spigot Pipe:

When testing large hub-and-spigot pipe, particularly reinforced concrete pipe, with three-edge bearings the determination of a center of application of load that will insure a fair test becomes a difficult problem. The hubs vary in shape and dimensions, with resulting variations not only in the length of the pipe that may be loaded, but also in the amount by which the stiffness of the unloaded hub exceeds that of the loaded barrel or spigot. The Society's sewer pipe specifications provide that the center of the application of load shall be at the center of the loaded length; the American Concrete Institute specifications for reinforced-concrete sewer pipe and the Joint Committee's specifications for reinforced-concrete culvert pipe provide that it shall be at the center of the over-all length of the pipe. When a reinforced concrete pipe of this type is loaded according to the Society's sewer pipe specifications it is nearly always possible to observe that the failure starts at the spigot end of the pipe. The Joint Committee provision is the result of an attempt to provide a loading that would be in compliance with their recommendation that the loading shall be such as will produce a uniform deformation throughout the full length of the pipe. The Society's sewer pipe specification results in no unfairness in tests of clay and plain concrete pipe, unless it be for the larger sizes of pipe (36-in. and larger), because the ratios between the results of tests with the different bearings were determined from the records of tests made under those specifications. However, these differences in loading must be observed in making tests and in comparing the results of tests under the different specifications.

Tests of Pipe Which are "Out-of-Straight":

In testing pipe that are "out-of-straight" the failure of the upper bearing to have uniform contact with the pipe throughout the loaded length is sometimes very noticeable. The lower bearing is not so readily visible, so that its uniformity or non-uniformity of bearing goes unnoticed. All of the standard specifications provide that in such cases the pipe shall be turned to the position seeming to give the fairest test. This obviates a part of the difficulty, except when testing oval pipe, or circular pipe with oval reinforcing, when the bearings must be along the elements at the ends of the long diameter of the ellipse of the pipe or of the reinforcing. Very few pipe are so nearly "straight" that the bearing appears to be uniform throughout its full length, so that the test is equally fair to all pipe of average good workmanship. It has been suggested that some softer material, as a strip of soft wood, of leather or rubber belting, or of felt, be placed between the pipe and the upper bearing. It seems probable that these "softer materials" would be more effective visually than in securing greater uniformity in the distribution of the load, and none of them makes any improvement in the lower bearing. The principal advantage of three-edge bearings is the greater convenience in testing. It does not now appear that the greater accuracy secured by either of these three suggestions would be sufficient to warrant the decrease in convenience that would result from their use.

Lower Three-Edge Bearing for Tests of Large Pipe:

Considerable difficulty has been experienced in testing large pipe, particularly in the longer lengths, with the lower three-edge bearings as described in the standard specifications for drain tile and sewer pipe. This bearing is so narrow that a large pipe must be placed very accurately or it will "roll" when the load is applied. The Joint Committee's proposed specifications provide that the strips of the lower bearing shall be so placed that the distance between their inside vertical faces is not less than ½, nor more than 1 in., per foot of diameter of pipe. This provision allows a lower bearing that is wide enough to make this bearing convenient for use with all sizes of pipe. If, in addition, a minimum spacing of one inch be specified, four bearings, with strips 1, 2, 3, and 4 in. apart, will provide for tests of all sizes of pipe up to 8 ft. in diameter. To be exactly fair to all sizes of pipe, the spacing of the strips of this bearing should bear a constant relation to the diameter, or to the circumference, of the pipe. However, the range of spacings provided for in the Joint Committee specifications is not large enough to produce any material unfairness. Although strict compliance with the drain tile or sewer pipe specifications permits only the one lower bearing, the use of the wider bearing will increase the accuracy of the tests of large pipe.

MEANING AND VALUE OF ABSORPTION TESTS

Although the absorption test is included in all standard pipe-test specifications, except those of the American Concrete Institute, its real meaning and value are subject to much discussion. If a pipe is satisfactory it must be strong enough to support the loads coming upon it and must be durable enough to maintain the required strength. The absorption test usually is considered as an index of those qualities that control durability. This use of the absorption test is clearly provided in the standard specifications for drain tile, under which the manufacturer, on appeal, may have the absorption limit for his product raised so long as his tile meet the requirements of the freezing and thawing test. Porosity, as determined by the absorption test, often is considered as an index of the permeability or the denseness of the material in the pipe wall. Porosity and permeability, or perviousness, are more nearly antonyms than synonyms. In nature the most porous soils, as clay with 50 per cent of pore space, are nearly impervious, while some less porous soils, as sand with 25 per cent of pore space, are very permeable. Although the pore spaces in the material of a pipe wall are not directly comparable to those in clay or sand, the permeability of concrete pipe is reduced and the percentage of absorption is increased when the proportion of fine aggregate is increased. The percentage of absorption is not necessarily an index of permeability, and should not be considered as a measure of durability on that basis. It seems very probable that the test would be of greatly increased value if a test method could be devised whereby the results would show not only the ultimate percentage of absorption but also the rate of absorption.

ABSORPTION TEST METHODS

Most controversies over absorption test results arise from differences of procedure in the two principal operations of the test: namely, drying and immersing. The absorption test as prescribed in all of the standard pipe test specifications has been criticized on the ground that drying is at too high a temperature and that immersion in boiling water is unfair to concrete specimens. Differences in procedure in these treatments will be discussed briefly.

Drying Treatment:

Each of the standard specifications requires that the absorption specimens be subjected to repetitions of the prescribed drying treatment till the weights after two successive treatments check closely. Under the drain tile and sewer pipe specifications the specimens are dried for 2 hours at a temperature of 110° C., allowed to cool to room temperature, and weighed. If a large number of specimens is being dried at once and check weights are taken for all of them, an incorrect dry weight will result from the fact that a major part of each 2-hour drying period will be required to heat the whole mass of specimens to the prescribed drying temperature. A more nearly true dry weight will be obtained if only three or four representative samples are used for determining check weights.

Some objection has been made to drying specimens from concrete pipe at 110° C., on the basis that this temperature drives off water of crystallization and that it will turn entrapped water to steam which will have pressure enough to break out the restraining pores and thus cause a higher absorption percentage. The absorption percentages for both clay and concrete specimens dried at different temperatures (between 25 and 180° C.) vary in almost direct proportion to the losses in weight in drying, indicating that if the actions predicted occur their effects are small. The data from such studies show also that the rate of loss-in-weight becomes so slow near the end of the drying period, particularly when the drying is at temperatures below 100° C., that a specimen may appear to be dry when it still contains an appreciable amount of moisture that can be driven off if drying at that temperature be continued long enough.

Immersion Treatment:

The 5-hour boiling immersion treatment produces a more nearly complete absorption than any other practicable treatment, and so far as reported data show does not give a result greater than that secured by immersion in water at room temperature if this immersion be continued long enough. The rate of absorption for some specimens is so rapid that they will attain their ultimate absorption in a short time; with others the rate is so slow that 2 to 3 months immersion in water at room temperature is required before the absorption percentage will equal that by the boiling test. The boiling test shows more nearly the ultimate absorption than can be shown by immersion in water at room temperature, unless the immersion period be much longer than is desirable in a test method.

If correct results are to be obtained in this test, the specified test procedure must be followed, particularly that stating that at the completion of the boiling the specimens must be cooled in water before weighing. If a specimen which has been immersed for a long time in water at room temperature be boiled as prescribed for this test and weighed immediately at the end of the boiling, its weight will be less than when the boiling was started; but if allowed to cool in water after boiling, its weight in nearly all cases will equal or exceed that before boiling.

DURABILITY TESTS

Freezing and Thawing Test:

The freezing and thawing test for drain tile is the only durability test that has been standardized. As originally recommended and adopted, this test provided that to be acceptable the specimens should withstand a given number of treatments consisting alternately of freezing, and thawing in boiling water. Studies made by subjecting whole tile to freezing out of doors in winter and thawing indoors at room temperature, showed that concrete tile that were almost unaffected by this natural test were very quickly destroyed in the artificial, accelerated test. Laboratory studies of this difference showed that these specimens and laboratory specimens (briquets and 2 by 4-in. cylinders) were so seriously affected by boiling that some of them lost 50 per cent of their structural strength in one boiling. On the basis of these data the procedure in the freezing-and-thawing test was changed to provide that the specimens should never be boiled, either during this test or in any preceding treatment. A similar effect, though less in extent, was found for specimens from some clay tile. The Iowa State Highway Commission has used this test extensively as a general factory test, by which they have determined an allowable absorption limit for the product from each factory. So far they have found that tile which meet a safe absorption limit as determined by the freezing-and-thawing test are satisfactory as to durability. These tile have not been subjected to any agency, except freezing and thawing, that would require a durability test. In general, durability against injury from freezing and thawing is a problem relating to certain qualities and classes of clay pipe. The freezing and thawing attack of well-made concrete pipe seems to be limited largely to those pipe made with aggregates which are in themselves subject to injury by freezing and thawing.

Chemical Tests:

Just as resistance to injury by freezing and thawing is primarily a problem of clay pipe, durability against attack by chemical-bearing soils and waters is primarily a problem relating to concrete pipe. Experience has shown that good-quality concrete pipe, and some clay pipe, are not durable in soils containing high concentrations of natural alkali salts. Some difficulties have been encountered also under the conditions found in certain industrial and sanitary sewers. Although several research agencies are working on this general problem, its phases are so numerous and so inter-dependent, and it is so much in controversy, that the adoption of standard specifications and tests must be deferred until more definite and conclusive research data are available.

THE A.S.T.M. SPECIFICATIONS FOR SEWER PIPE AND THE FIBER STRENGTH OF THE MATERIAL

By F. B. Lysle[1]

Synopsis

The paper embodies a discussion of the A.S.T.M. Specifications for Clay Sewer Pipe, the A.S.T.M. Specifications for Cement-Concrete Sewer Pipe, the U. S. Bureau of Public Roads specifications for pipe for highway work, and the relation of these specifications to the fiber strength of the materials used. The proposal is advanced that a consideration of the matter herein set forth might lead to a revision of the specifications.

Some time during the past year while looking over quite an accumulation of data on the breaking load of pipe, both vitrified-clay pipe and cement-concrete pipe, it was noticed that certain sizes of pipe showed a tendency to break at relatively lower loads than pipe of other sizes. In seeking some reason for this, it was decided to investigate the A.S.T.M. Specifications for Clay Sewer Pipe (C 13 – 24)[2] and for Cement-Concrete Sewer Pipe (C 14 – 24)[3], in their relation to the formula for fiber strength, which appears in the A.S.T.M. Specifications for Drain Tile (C 4 – 24).[4] There are two formulas given in the Specifications for Drain Tile, as follows:

$$M = 0.20\, r \frac{W}{12} \quad \text{(1)}$$

$$f = 6 \frac{M}{t^2} \quad \text{(2)}$$

These may be combined into one formula:

$$f = 0.10\, r \frac{W}{t^2} \quad \text{(3)}$$

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[1] Formerly Resident Engineer, Eastern Clay Products Association.

[2] 1927 Book of A.S.T.M. Standards, Part II, p. 168.

[3] *Ibid.*, p. 179.

[4] *Ibid.*, p. 190.

and for further convenience in using, this may be transformed into the following:

$$f = \frac{m}{2}\left(\frac{D + t}{t^2}\right)W \qquad \text{(4)}$$

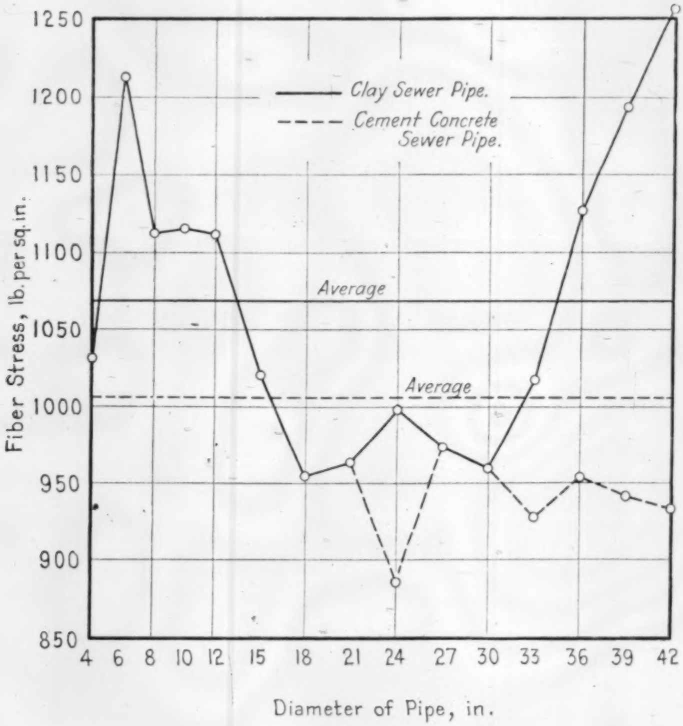

FIG. 1.—Fiber Strength of Sewer Pipe According to A.S.T.M. Specifications.

where f = fiber strength (modulus of rupture), in pounds per square inch;
m = a constant = 0.10;
D = internal diameter of barrel of pipe, in inches;
t = thickness of barrel of pipe, in inches;
W = load in pounds per foot of length (sand bearings).

The only new quantities which have been used are D, the internal diameter of the barrel of the pipe, which is also used to designate the nominal size of the pipe, and the quantity m, which is a combination of all the numerical constants appearing in the original expressions.

When the loads and dimensions, which are set forth in the standard specifications for both clay sewer pipe and cement-concrete sewer pipe, are substituted in the formula, there is obtained for each size of pipe the "designed" fiber strength. The results are shown graphically in Fig. 1.[1] It will be noticed that the average, maximum and minimum values for the clay pipe are all slightly higher than those for concrete pipe. But what is more important is the variation shown in the values. For clay pipe the maximum value is 32 per cent higher than the minimum; the maximum is 18 per cent higher than the average; the minimum is 11 per cent lower than the average. For concrete pipe the maximum value is 37 per cent higher than the minimum; the maximum is 21 per cent higher than the average; the minimum is 12 per cent lower than the average.

It will also be of interest to compare the A.S.T.M. specifications for clay sewer pipe with the tentative specifications of the U. S. Bureau of Public Roads for pipe to be used in Federal Aid road work. The U. S. Public Road tentative specifications impose a load of 2250 D (sand bearing) as the minimum load which the pipe must stand, where D is the diameter of the pipe in feet. The relation between these two specifications is shown graphically in Fig. 2. Some rather remarkable results are developed here, in the U. S. Public Roads specifications. The maximum value of fiber strength is 264 per cent higher than the minimum; the maximum is 38 per cent higher than the average; the minimum is 62 per cent lower than the average. Another very remarkable fact shown is that, in sizes from 12 to 30 in., inclusive, the fiber strength demanded by the U. S. Public Roads specifications is the same value, namely, 1462 lb. per sq. in. This fiber strength also applies to concrete pipe in all sizes from 12 to 42 in., inclusive, except the 24-in. size, when the U. S. Public Roads loads are used. This peculiarity is due to the fact that the ratio of thickness of barrel to diameter of pipe is constant for these sizes of pipe, as will be explained later in this paper.

In each of the A.S.T.M. specifications for sewer pipe there will be found a table of permissible variations. An examination of this table will show that the diameter of the barrel may be increased by a certain amount, and the thickness of the barrel may be diminished by a certain amount. If these differentials are used as indicated and the corresponding fiber strengths are calculated, there will be obtained the maximum fiber strengths demanded of the different sizes of pipe. These are shown in Fig. 3 in comparison with the fiber strengths calculated for clay pipe of the standard dimensions. Here again a wide variation is shown in the values obtained. The maximum fiber strength is 41 per cent higher than the minimum; the maximum is

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[1] The values are identical on all diameters up to 30 in., excepting the 24-in. diameter.

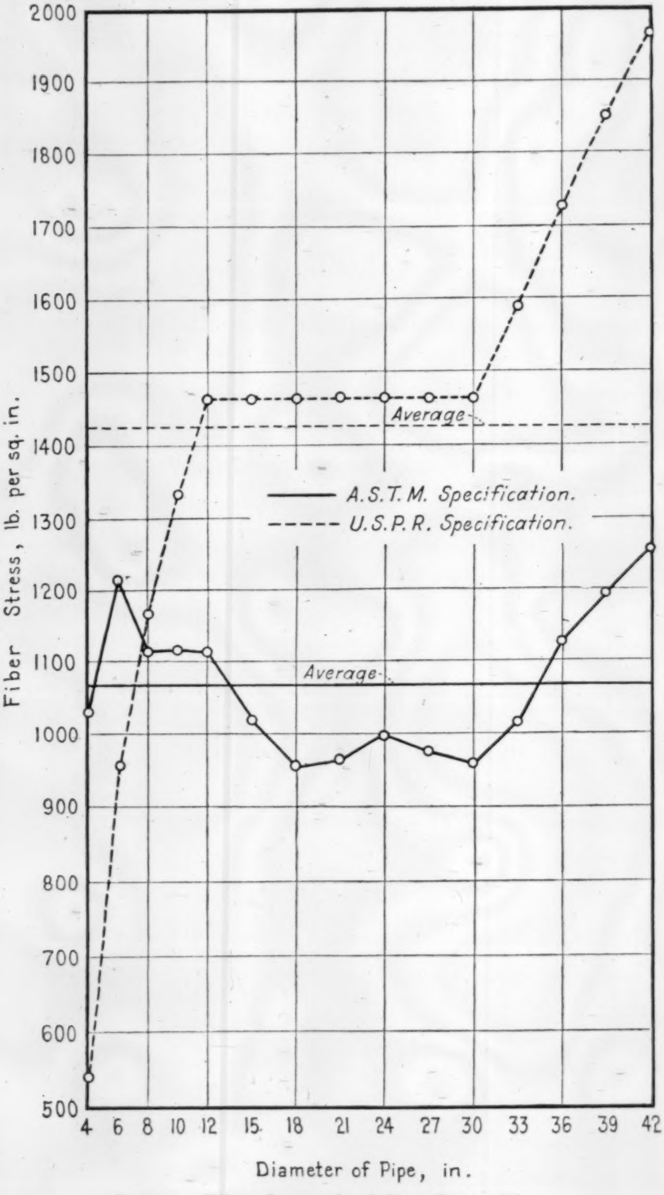

FIG. 2.—Fiber Strength of Clay Sewer Pipe.

21 per cent higher than the average; the minimum is 14 per cent lower than the average. A similar variation may be obtained for the concrete pipe.

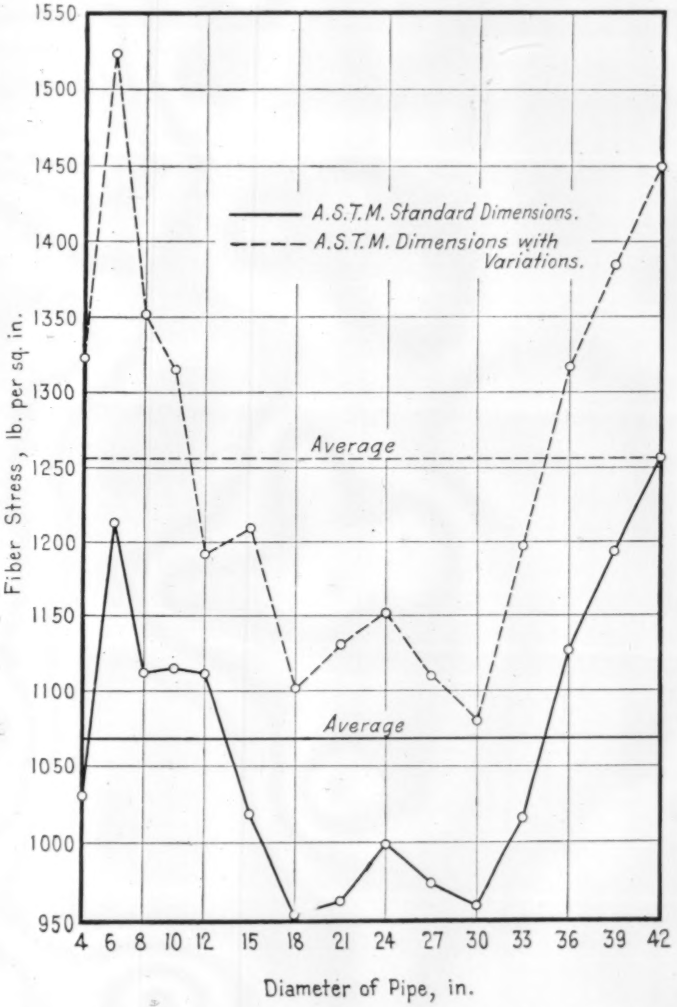

FIG. 3.—Fiber Strength of Clay Sewer Pipe, According to A.S.T.M. Specifications.

Let us now return to Eq. 4 and see what information can be gathered from a consideration of the algebraic relations set forth. Let us assume some constant fiber strength for the material, say 1000 lb. per sq. in. Then a plot of the load (W) against the thickness of the barrel (t) will give a parabola for a definite diameter of the barrel (D). Using all the diameters will give a family of parabolas as shown in Fig 4. Such a graph as this will show how to design the thickness of the barrel of the pipe to obtain a definite load-bearing value for the pipe for an assumed fiber strength. Similarly, a plot of load against diameter of pipe will give a family of hyperbolas for various thicknesses of barrel. This latter graph will not be quite as useful for design as the former one.

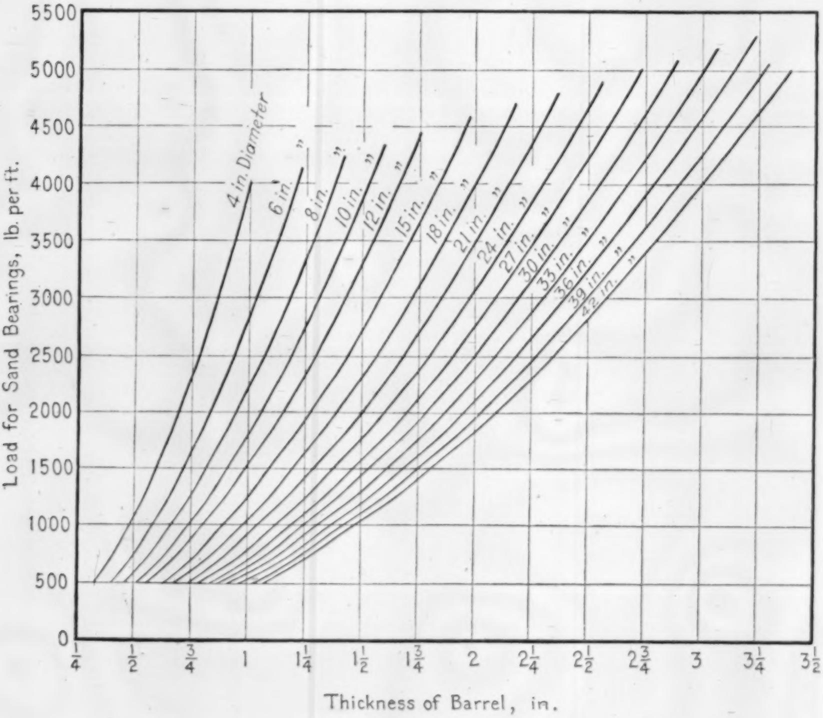

FIG. 4.—Curves for Design of Sewer Pipe, Based on Fiber Strength (Modulus of Rupture) of 1000 lb. per sq. in.

If in Eq. 4 we let $t = bD$, in which b is the ratio of thickness to diameter, and make this substitution, the equation will transform into:

$$f = \frac{m}{2}\left(\frac{1 + b}{b^2}\right)\frac{W}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Now, if a constant fiber strength of 1000 lb. per sq. in. is assumed, and if b is assumed a constant, say $\frac{1}{12}$, then the load will be directly proportional to the diameter. Or to state the matter in another way, if the load is proportional to the diameter, as in the U. S. Bureau of Public Roads specifications, and the thickness of the barrel is a constant fraction of the diameter of the barrel, then the fiber strength will be constant. The property was developed above in the paragraph in which the U. S. Bureau of Public Roads specifications were discussed.

CONCLUSIONS

The strength specifications for clay sewer pipe or concrete sewer pipe may be approximated by several methods:

1. By the arbitrary assumption of safe loads derived from an accumulation of test data;
2. By assuming a definite unit fiber strength and designing the thickness of the barrel and the load accordingly;
3. By assuming loads from certain heights of trench fill and designing the pipe to fulfil the conditions.

The present A.S.T.M. specifications were evidently arrived at by the first method. The third method could not be readily followed, unless the load requirements were made great enough for the deepest trench fill, and in this case the pipe would be uneconomic for a majority of its uses. The second method would seem to be the one which should be used.

Acknowledgment.—Acknowledgment is made to H. C. Peirson, formerly Testing Engineer, Eastern Clay Products Association, for assistance in preparing data.

DISCUSSION

Mr. Bateman.

MR. JOHN H. BATEMAN[1] (*presented in written form*).—The writer agrees with Mr. Lysle's conclusion that the present A.S.T.M. Specifications for Cement-Concrete Sewer Pipe (C 14 – 24) are inconsistent with reference to the relation between the crushing strength and the wall thickness of the pipe. The use of the formula given in Eq. 4 of the paper, however, will give fiber strengths much in excess of the known values for concrete. The writer recently had the opportunity to compare the crushing strength of machine-made concrete sewer pipe with the compressive strength of the concrete used. The following tabulation gives the data secured:

DIAMETER OF PIPE IN.	WALL THICKNESS, IN.	CRUSHING STRENGTH (SAND BEARINGS), LB. PER LIN. FT.	CALCULATED FIBER STRESS, LB. PER SQ. IN.
12	$1\frac{1}{8}$	2750	1443
15	$1\frac{3}{8}$	2710	1180
18	$1\frac{1}{2}$	2670	1155
24	2	3600	1170

The compression specimens of the concrete used were made by tamping the concrete in thin layers in 5 by 10-in. cylindrical molds. Tamping was performed with wooden blocks in an attempt to compact the concrete in the molds to a degree equivalent to the action of the machine which made the pipe. The average compressive strength of the concrete was 4916 lb. per sq. in. at 28 days. Flexure specimens were not made because of the difficulty of compacting the concrete, so the direct flexural strength of the concrete was not obtained. Abrams[2] proposed the use of the following formula for calculating the flexural strength from the compressive strength:

$$R = 0.29\,S - 0.000032\,S^2$$

where R = modulus of rupture, in pounds per square inch; and
S = compressive strength, in pounds per square inch.

Substituting the value of 4916 lb. per sq. in. in this formula the flexural strength is found to be 653 lb. per sq. in. The calculated fiber stresses given in the above table are entirely too high, even in view of

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[1] Research Professor of Highway Engineering, Louisiana State University, Baton Rouge, La.

[2] D. A. Abrams, "Flexural Strength of Plain Concrete," *Proceedings*, Am. Concrete Inst., Vol. XVIII (1922).

Mr. Bateman. the fact that the concrete used for machine-made pipe usually has very high strength.

Messrs. Marston and Schlick. MESSRS. ANSON MARSTON[1] AND W. J. SCHLICK[2] (*presented in written form*).—This paper presents some very interesting comparisons, particularly those showing that the strengths specified in the Society's sewer pipe specifications do not increase in a logical order as the sizes of pipe increase. There are, however, three points in the author's presentation which may lead to an incorrect appreciation of this general problem.

The moment equation used is the theoretical equation for the bending moment in an elastic ring under similar and opposite loads, with the value of the constant determined empirically from the results of tests of clay and concrete pipe with sand bearings and of tests of curved beams cut from those pipe. The value W in that equation is the supporting strength of the pipe when tested with sand bearings. This strength is very close to that which the pipe would have developed with "ordinary" pipe-laying, but with any other pipe-laying method the supporting strength developed in the trench will be greater or less than W—greater for improved earth, sand and concrete-cradle beddings and less for "impermissable" beddings. If the test load requirements are to be based upon the loads for certain heights of trench fill, the supporting strengths necessary for certain specified average construction conditions must be determined, and then the test strengths necessary with each of the standard test bearings must be so correlated with these that pipe which meet the test requirements will be safe under those conditions.

The requirements of the standard specifications for minimum test loads and for minimum wall thicknesses very logically may be based upon different considerations. The test load requirements should be those which will insure that pipe meeting them will have the desired strengths. The minimum wall thicknesses are fixed upon other bases, and on those bases they logically may be less than those with which any manufacturer can meet the strength requirements.

The use of the author's suggestion for determining strength requirements "by assuming a definite fiber strength and designing the thickness of barrel and the load accordingly" may lead to considerable difficulty. If a series of clay or concrete pipe of varying diameters and wall thicknesses be made of material of a constant strength, the moduli of rupture determined from the test loads by the formulas given will vary with some function of the diameter and the wall thickness. With the wall thicknesses ordinarily used, the value of

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[1] Dean of Division of Engineering, Iowa State College, Ames, Iowa.

[2] Drainage Engineer, Engineer Experiment Station, Iowa State College, Ames, Iowa.

Messrs. Marston and Schlick.

the modulus will decrease as the size of the pipe increases, though not in direct proportion. If the strengths be determined from the correct value of the modulus for an intermediate size of pipe, these strengths will be less than the test strengths for the small sizes and greater than the test strengths of the large pipe. These formulas are so simple to use that it is unfortunate that the actual relationships between the various quantities do not follow those of the formulas.

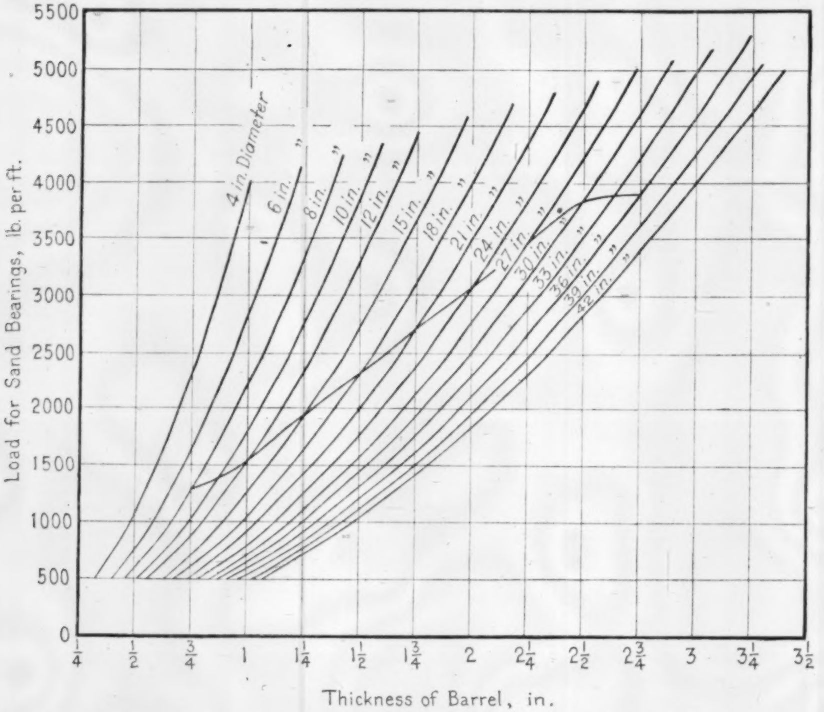

FIG. 1.—Curves for Design of Sewer Pipe, Based on Fiber Strength (Modulus of Rupture) of 1000 lb. per sq. in., with Intersecting Curve Showing Requirements of A.S.T.M. Specifications for Clay Pipe.

Mr. Linton.

MR. ROBERT LINTON[1] (*presented in written form*).—The paper presents in a clear and logical manner some very interesting data bearing on the question of sewer pipe and with the laudable purpose of developing specifications which will not rest solely on empirical bases. His graphs show that pipe in size ranging from 15 to 33 in. in diameter have a fiber strength below the average, from which it might be deduced that he regards the shell thickness in these sizes too low.

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[1] Vice-President and General Manager, Pacific Clay Products, Los Angeles, Calif.

Mr. Linton. He also makes comparisons between the A.S.T.M. Standard Specifications and those of the U. S. Bureau of Public Roads. The latter call for shell thicknesses greater than the A.S.T.M. requirements, but the writer is of the opinion that the minimum load requirements of the U. S. Bureau of Public Roads are considerably higher than ordinary practice requires. The A.S.T.M. specifications, while derived in an empirical way, are based on tests and actual service conditions of pipe in use over long periods of years and should, therefore, serve as a safe guide for the design and construction of pipe sewers.

The suggestion is made that specifications should be based on the method of assuming a definite unit fiber strength and designing the thickness of the barrel and load accordingly. In view of the wide range of fiber stresses for different sizes as shown in his graphs, it would seem that any application of the fiber stress method should be preceded by very comprehensive tests, both in the laboratory and field.

The writer has taken the parabolic graphs shown in Fig. 4 of Mr. Lysle's paper, plotted thereon an intersecting curve showing the thickness requirements of the A.S.T.M. Specifications for Clay Pipe (C 13-24) and requirements for load tests of pipe 8 to 36 in. in diameter. This curve follows a quite uniform increase up to 30 in., where it bends to a nearly horizontal direction for the 33 and 36-in. sizes. However, the fiber stresses plotted in Fig. 1 of the paper show materially greater strength for 33 and 36-in. pipe than for 30-in., and have shown adequate strength in service. It would appear then that pipe thicknesses in the larger sizes have been standardized to figures which are ample for load-bearing purposes.

Mr. Lysle. MR. F. B. LYSLE[1] (*author's closure by letter*).—The criticisms presented in the paper approach the matter from different viewpoints, and all the matter which has been offered in the criticisms is very valuable and widens the horizon of the general consideration of the subject. The author desires to draw attention to the theme of his paper, that the fiber strengths of the material varied quite widely for the various sizes of pipe, when these fiber strengths were calculated by the proposed equation. It was assumed that the equation was valid, although the author realized that the constant in the equation varied about 66 per cent in value from that proposed by A. N. Talbot in his theoretical treatment of the problem. Neither was the question considered whether a 1200-lb-per-sq-in. modulus of rupture for concrete was approximately twice as large as it should be. Neither were the values from trench loadings considered.

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[1] Formerly Resident Engineer, Eastern Clay Products Association.

Mr. Lysle. When one breaks several hundred pieces of pipe of a particular size from different plants and finds that the values average close to the A.S.T.M. load requirements and then breaks several hundred pieces of pipe of another particular size from different plants and finds that these values average well above the A.S.T.M. load requirements, then the conclusion must be drawn that one size is under-designed or the other size is over-designed, as it is fair to assume that the average intrinsic value of the material in the two different sizes should be approximately the same. This is what happened actually, and the formula points out the lack of uniformity in design.

THE HOW AND WHY OF GASOLINE PERFORMANCE

By J. Bennett Hill[1]

Synopsis

Opinion as to what constitutes a good motor gasoline has changed very materially in the last few years. The question is now being considered in the light of what the automobile engine demands for optimum performance and of the limitations placed on the material by transportation, handling and storage requirements, rather than in the old light of what appealed to unenlightened public prejudice.

The properties required by the engine are primarily ease of starting in a cold motor, normal functioning without choke at the lowest possible temperature, uniform feed from the carburetor, and combustion under high cylinder pressures with minimum detonation. These points can be suitably covered by volatility and detonation tests. Other characteristics such as odor, corrosive qualities of the fuel itself or of the exhaust gases, crank case dilution and formation of solid deposits are important secondary requirements of the automobile. Handling, transportation and storage require in addition that the fuel have a minimum evaporation loss, that it be not subject to chemical or physical change on long standing, and that it be as free as possible from any harmful physiological effects from contact with it. Practically all of these requirements can be covered by the laboratory tests which are now used to determine gasoline quality.

It is interesting to note that color and gravity, the criteria of former times, are conspicuously absent from these significant tests.

Since the development of the automobile over the last twenty-five years, gasoline has risen from a position of a waste by-product to become one of the leading manufactured commodities produced in this country. The gasoline tonnage of the United States is now about three-quarters that of lumber, nine-tenths that of steel, one and a quarter times that of cement and two and one-half times that of brick. It is estimated that more than 50 per cent of the power developed in this country last year was developed by gasoline and that automobiles under gasoline propulsion traveled about forty times as many miles as the railroad trains and trolley cars together.

But on account of its bearing on the individual comforts of the fifty million odd drivers in this country, gasoline is a much more discussed topic than even these figures would indicate. Listen to a group of men around the dinner table. The subjects of the steel that enters into their automobiles or the bricks that enter into their houses are seldom discussed, but the gasoline that drives their cars is a subject that is not often left untouched. In fact, gasoline is taking that place in men's conversation that the subject of clothes traditionally takes in that of our wives and daughters. Some of the gasoline information which is commonly passed around in this way is correct, but probably the great bulk of it is misinformation. It is, therefore, quite appropriate that we devote a portion of our time in discussing gasoline, in the light of the best information which we have on it to-day.

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[1] Chief Chemist, The Atlantic Refining Co., Philadelphia, Pa.

The first question which we have to ask ourselves is, What is expected of gasoline? what are its important requirements? From the user's point of view the primary consideration is how well it will drive the car. He expects from it easy starting in cold weather without excessive use of the choke while the engine is warming up, good acceleration, even carburetion in very warm weather, combustion at low speeds and open throttle without the disagreeable knock or detonation, and maximum mileage. He further requires, as what might be called secondary considerations, that the gasoline shall not have any direct or indirect harmful effect on his car or its occupants. for example, that it shall not produce an objectionable odor in the car, that it shall not corrode the gasoline feed system or leave deposits which cause the delicate parts of the carburetor or of the vacuum tank to stick, that it shall when burned produce exhaust gases which are not corrosive to the motor, and that it shall not under ordinary running conditions excessively dilute the crank case oil and destroy its lubricating properties.

The man who handles gasoline or transports it has in addition his own requirements. A high evaporation loss in storage or handling makes a product undesirable to him. He must avoid gasolines that are chemically unstable and which, while they may be satisfactory when he receives them, will develop objectionable properties while in his hands. Again, he may have the type of dispensing equipment in which the gasoline is stored over water and would object strenuously to the fuel containing constituents which might be so dissolved out. A poisonous nature of certain constituents will require him to take special precautions with which he may not care to be bothered. If he can get a gasoline which meets these various ideas he will naturally take it in preference to one that will not.

All of these points the petroleum refiner must try to meet when he supplies gasoline for automobiles. The question naturally arises: How are he and the consumer to recognize good gasoline? Many try to answer the question by putting the product under question in one or several cars and getting the reactions of the drivers. When a specially equipped test car is used, under definite conditions, over a well-known course, and in the hands of an experienced automotive engineer, this method is very satisfactory for certain points, but most people interested are not so equipped. It seems to be universal experience that, where less carefully controlled tests have been run, the reported results are so widely variant due to the personal factor of the drivers or to some unknown and unexpected variable that they are practically worthless. There have been obtained, however, under carefully controlled conditions, considerable fundamental data which enables us to correlate the chemical and physical tests of motor gasoline with its performance, so that by inspection of these tests it is possible to predict fairly well what performance the motorist may expect.

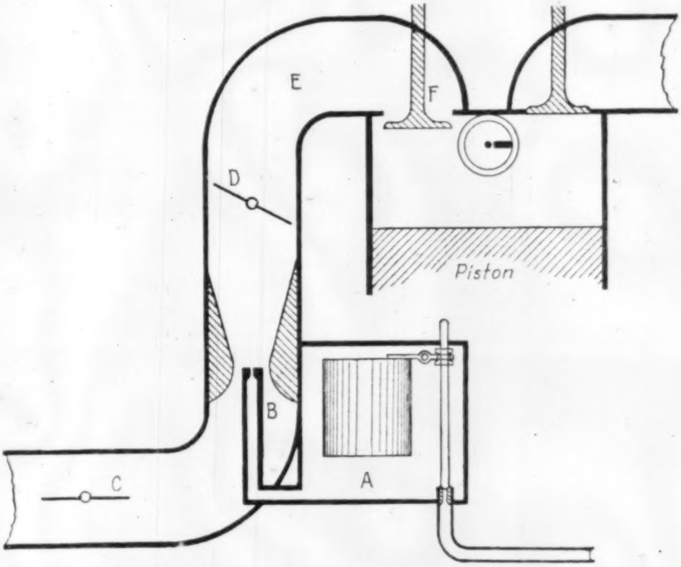

FIG. 1.—Diagram of Engine Intake System.

In order to discuss more clearly these individual requirements of motor gasoline a brief review of what takes place in the fuel intake system of an automobile engine may not be amiss. Such a system is represented diagrammatically in Fig. 1. The liquid gasoline, fed from the main tank or from the vacuum tank, is delivered to the bowl, *A*, of the carburetor, its flow being controlled by the float valve so as to keep a constant head in this chamber. Considering only the simplest design of carburetor, the bowl is connected with the jet, *B*, which protrudes into the air stream. The air passing through the carburetor picks from the jet, by reason of its velocity, a spray of gasoline, carefully metered by the size of the orifice in the jet and carries it along. The air intake to the carburetor is provided with the choke valve, *C*, a butterfly valve, which is normally wide open and offers no resistance to the free entrance of air. Just beyond the carburetor on the engine side is placed the throttle valve, *D*, which in its normal position is nearly closed. Through this the mixture of gasoline and air passes into the intake manifold, *E*, and thence into the various cylinders as the intake valves, *F*, open and the receding piston creates the suction to draw it in. With the throttle in the closed or idling position the pistons are required to suck against the closed valve and a vacuum is created in the intake manifold. The charge which is drawn into the cylinder at sub-atmospheric pressure is therefore low in weight, and low in energy on explosion, allowing the motor to develop only a low torque. As the throttle is opened the vacuum in the intake manifold is decreased due to decreased resistance to the passage of incoming mixture from the carburetor, the charge received by the cylinder is correspondingly heavier, more energy is produced and the torque is increased. The carburetor is so designed that the mixture ratio, that is the ratio of the weight of air to the weight of gasoline, remains roughly constant at say 12 to 1 regardless of the quantity of air drawn through the carburetor. In starting the car in cold weather it is necessary to supply a richer mixture (a lower air-fuel ratio) than this and the choke is therefore provided. Pulling out the choke closes the valve on the intake side of the carburetor and so causes the vacuum of the intake manifold to extend back into the carburetor. This vacuum acts on the jet and actually sucks additional gasoline into the mixture. Chokes are so designed that the air-fuel ratio can be decreased to 2 to 1 or even 1 to 1.

Starting Qualities:

This brings us to the first requirement of the motorist of his gasoline, which is that it shall give easy starting in a cold motor. In order that the charge may fire in the cylinder it is necessary that the mixture drawn in be richer than a certain maximum air-fuel ratio. While the carburetor, without choking, is set to deliver such a mixture, the bulk of the gasoline in the cold engine is not volatilized but remains in the form of a liquid spray and settles on the manifold walls, so that the mixture actually reaching the cylinder contains only the volatilized portion of the fuel plus some entrained liquid. It is much leaner than the required ratio and therefore will not fire. Easy starting, therefore, depends upon the volatility of the gasoline since it is evident that, even if the artificially enriched mixture is produced by the use of the choke, enough of the gasoline must vaporize at the temperature of the manifold to give a combustible mixture. For example, if the mixture required by the cylinder is 12 to 1 and the mixture supplied by the choked carburetor is 2 to 1 it is readily seen that approximately one-sixth of the gasoline must be vaporized under these conditions in order to reach the cylinder and start the engine. Bridgeman and Cragoe[1] have correlated this volatility of the first portion of the gasoline with the data obtained by the A.S.T.M. distillation test and have shown that from the low points on this test the starting quality of the gasoline may be predicted. A reference to Fig. 2 will bring out the meaning of this more clearly. The curves represent distillation tests of gasolines determined by the A.S.T.M. method, the ordinates being the vapor temperatures on the test and the abscissas the percentages distilled over at these temperatures. Curve *A* represents the average motor gasoline marketed in this country in the winter of 1926–1927 as shown by the Bureau of Mines Semi-Annual Motor

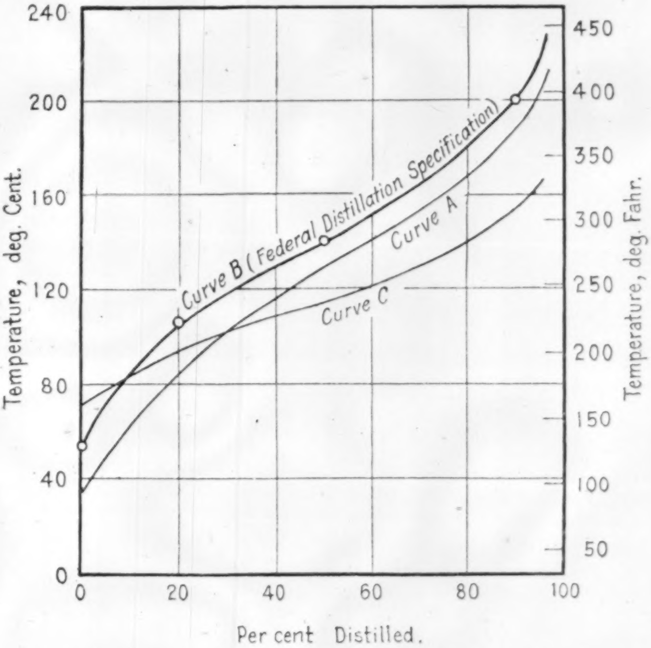

FIG. 2.—Curves Representing Distillation Tests of Gasolines.

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[1] Am. Petroleum Inst. *Bulletin*, Vol. IX, January 31, 1928, p. 54.

Gasoline Survey.[1] Curve B is the Federal distillation specification for motor gasoline. Curve C represents a hypothetical gasoline which is more volatile over the greater part of its range than gasoline A. The graphs show, however, that gasoline A would be easier starting in cold weather than gasoline C on account of its higher volatility on the first part of the curve. The gasoline manufacturer must therefore see to it that his product is sufficiently volatile, as indicated by the front end of the A.S.T.M. distillation curve, to give minimum starting troubles in cold weather.

Choking:

Having started his engine, the motorist is next bothered by the fact that he must keep his choke part way out until the motor warms up. His second requirement of the fuel, therefore, is that this trouble shall be reduced to a minimum and that he be able to push in the choke and reach normal operation of the engine in the shortest possible time. This may seem on first glance to be the same problem as that of starting, and it is a question of the volatility of the fuel. A little closer study shows, however, that the volatility requirements are rather different. After the first few seconds of operation the liquid gasoline deposited on the manifold walls begins to reach the cylinder, as liquid. Part of this vaporizes in the cylinder serving to enrich the mixture and to eliminate the necessity of providing such a highly enriched mixture from the carburetor. Experience indicates, however, that in order for the engine to function properly and to develop maximum torque, it is necessary that a minimum of about 60 per cent of the gasoline required by the cylinder reach it in the form of vapor, after volatilization in the intake manifold. Assuming again that the unchoked carburetor delivers a 12 to 1 mixture and that this ratio is required for proper engine operation, the choke can be pushed in only when the intake manifold has warmed up to such a temperature that 60 per cent of the gasoline is vaporized. The more volatile the gasoline, therefore, in its middle fractions the shorter will be the warming up period of the engine. This behavior may again be correlated with the intermediate points on the A.S.T.M. distillation curve and warming up performance may be predicted from these data.

Acceleration:

The third requirement of the motorist, acceleration, is also related to the volatility of the gasoline. When the motorist is driving along at nearly closed throttle and, therefore, low manifold pressure and suddenly steps on the accelerator, the immediate effect is to raise the pressure in the intake manifold. If the gasoline is of low volatility so that just enough vaporized at the low pressure, the increase in pressure will throw out part of the gasoline as liquid and the engine will not receive the proper mixture. Quick acceleration, therefore, requires a sufficiently high volatility to permit the mixture to readjust itself immediately to the changed pressure conditions. Part of the burden of acceleration has been lifted from the gasoline in recent years by improved carburetor design which provides an accelerating well containing a small reserve supply of gasoline, which is thrown into the mixture when conditions are upset by the pressure rise.

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[1] U. S. Bureau of Mines Report of Investigations No. 2795, February, 1927.

The three points just discussed might give the impression that the greater the volatility of gasoline, the better. As a matter of fact this is true only up to a certain point. Recent advances in engine and carburetor design have minimized the troubles resulting from low volatility, so that a good commercial gasoline to-day will operate just as satisfactorily as a special product of much higher volatility. The occasional demand for much more volatile fuels is unjustified, especially in view of the fact that it is only by holding to present standards of volatility that the petroleum industry has been able to meet the enormously increased demand for gasoline at prices favorable to the motoring public. It is worthy of note, however, that the last five years have seen a considerable increase in the volatility of commercial gasolines. A comparison of curves *A* and *B* of Fig. 2 will show how much more volatile commercial gasoline is than that called for by the Federal specification.

One requirement of the motorist is actually limiting on the volatility of gasoline. This is the elimination of the trouble commonly known as gas lock. In very warm weather or in engines designed so that the carburetor becomes heated, the gasoline may actually start to boil in the passages leading to the jets. Feeding a mixture of vapor and liquid to the jets naturally completely upsets their metering, with the result that the mixture ratio is frequently thrown off so far that the engine refuses to operate. This difficulty can be avoided by regulating the amount of very volatile material in the gasoline. The likelihood of trouble from a gasoline may be predicted by a study of the initial part of the A.S.T.M. distillation curve.

Knocking or Detonation:

One of the most discussed properties of gasoline to-day is its susceptibility to knocking or detonation. The exact cause of detonation is still unknown. It is apparently an abnormal course of combustion resulting in rapidly developed high pressures and manifesting itself in the sharp metallic ping which so annoys the average motorist. The significant point in the occurrence of detonation is that the tendency to detonate increases with the pressure at which the mixture is fired. Tendency to knock becomes less as the speed of the engine increases. Detonation is observed, therefore, when the throttle is opened at low speeds, since a charge at higher pressure is taken into the cylinder. This is submitted to the normal compression ratio and gives a higher final pressure. The tendency to detonate also varies with the chemical composition of the fuel so that while one gasoline may detonate under very mild conditions, a second fuel may detonate only when subjected to very extreme conditions. The only satisfactory method so far developed for rating the knocking tendency of a fuel is by actual engine test under carefully controlled conditions. Fuels may be rated in terms of the maximum compression pressure they will stand without detonating, or some related factor. Since benzol is a very good anti-knock fuel, anti-knock property is frequently described by reporting the gasoline in terms of the percentage of benzol which must be added to a bad-knocking fuel in order to make it equal the unknown gasoline.

Mileage:

A property of gasoline which receives far more attention from the lay motorist than it deserves is mileage. Leaving out the benzol and alcohol blends and considering only gasoline in its various qualities, it can be accurately stated that, pound for pound, all gasolines give practically the same energy on burning and that they are, therefore, capable of giving identical mileages. Due to the slight range of variation in the specific gravities of gasolines, a product with a high specific gravity (low test) should give somewhat higher mileage per gallon than a low specific gravity (high test), but the differences are not substantial. The prevalent belief that there is a difference between the mileages of fuels is founded in part on the fact that the motorist seeks to compensate for the hard starting and slow warming up of a low-volatile fuel by enriching his mixture at the carburetor and so sacrifices mileage in an attempt to mask qualities which are inherent in the fuel. He then makes the statement that this gasoline gives poorer mileage than some other product which is more volatile and for which he uses a more nearly correct mixture ratio.

Secondary Requirements:

While the properties that have been discussed are those which directly concern the operation of the engine, there are others which affect the motorist less directly but in which he is rightly interested. One of these is the question of odor, a matter which has no bearing on the functioning of the motor and yet one which contributes considerably to the happiness or unhappiness of the car user. It takes surprisingly little of an evil smelling gasoline to make the interior of a closed car distinctly objectionable. Unfortunately, no success has been attained in developing a mechanical nose to standardize odors. Inasmuch, however, as most of the malodorous compounds in gasoline are sulfur compounds belonging to the mercaptan group, the common "doctor" test which shows up this class of compounds is valuable as an odor requirement.

Another important secondary consideration of the motorist is freedom of the gasoline from tendency to corrode the finer parts of the intake system or to form deposits which may cause them to stick. Some poorly refined gasolines occasionally cause such trouble but adequate refining will insure against them. Corrosive substances in a gasoline are very readily detected by the A.S.T.M. copper strip test for corrosion.

The corrosion of engine parts due to combustion gases is quite a different matter. The main cause of such corrosion is excessive sulfur in the gasoline which burns to sulfur dioxide, leaks past the piston rings and forms sulfuric acid with any water in the crank case oil. This may produce serious corrosion of wrist-pins, timing chains, etc., within the crank case. The gasoline producer is very active in keeping sulfur down to safe limits and comparatively little trouble results. The quantitative determination of sulfur in gasoline by the A.S.T.M. method is easy, and excessive sulfur may, therefore, readily be detected. Recent developments in engine design involving ventilation of the crank case and elimination of water should practically eliminate such sulfur corrosion, irrespective of the sulfur content of the fuel, since dry sulfur dioxide has little, if any, corrosive effect. The question of pitting of exhaust valves is still a controversial matter, but it is the author's opinion that this is due not to corrosion but to errosion and that it is, therefore, not related to the quality of the gasoline.

The last important secondary requirement of the motorist is absence of excessive crank case dilution. A gasoline with a high-boiling, low-volatile "heavy end" is apt to give incomplete vaporization. The low-volatile material enters the cylinder as liquid and dissolves in the oil film on the walls where it remains unburned and eventually works down into the crank case oil, diluting the oil and cutting down its viscosity and, therefore, its lubricating value. All commercial gasolines will give this effect to some extent, particularly if the motor is used for short runs and requires many cold starts. On long runs where the motor is heated up for extended periods of time, the dissolved diluent is re-evaporated from the crank case oil so that an equilibrium dilution is normally reached. Work done by the Bureau of Standards has shown that the dilution attained may be regulated by controlling the volatility as shown by the last part of the A.S.T.M. distillation curve. There has been a trend in recent years to improve this property of motor gasoline very markedly.

Special Requirements:

The special requirements of handling, transportation and storage are not of concern to the motorist, but are, nevertheless, important considerations. Gasolines containing considerable quantities of very low-boiling hydrocarbons will show high evaporation losses in spite of precautions to minimize them. The refiner has come to realize the folly of inviting such losses and is removing these highly volatile portions or stabilizing the gasoline before shipping. These types of compounds generally appear as "loss" on the A.S.T.M. distillation test and their presence is, therefore, easily recognized.

Chemical instability may result in the product going off odor or forming gummy deposits in storage. It is generally the result of inadequate refining.

Harmful physiological effects from contact with a gasoline are apt to be a deterrent to its popularity due to the dangers of handling. One of the objections to aniline as an anti-knock agent is its poisonous character in the proportion in which it would have to be used in gasoline to be effective.

It is interesting to note the change in the old ideas. It was not so very many years ago when gasoline was judged entirely by two properties: color and gravity. It is now recognized that neither one of these has any bearing whatsoever on gasoline quality. The public formerly demanded a water white gasoline and felt that otherwise it was buying a poor product. It is now coming to realize that water-white color and good anti-knock quality do not often go together and that on many gasolines in order to make a colorless product the refiner must use expensive processes or sacrifice part of the anti-knock value, and that, furthermore, the color has no harmful effect on the use of gasoline. So many of the anti-knock gasolines are now artificially colored that the motorist has actually come to associate colored gasoline with good gasoline. This idea is, of course, just as fallacious as the prejudice in favor of a colorless fuel. Gravity appears also to have been completely dropped. Replacing these two criteria, we have raised in their stead another pair, volatility and detonation, which seem now to tell us practically all we need to know about the quality of motor gasoline.

The further improvement of motor performance is in the hands of both the motor car manufacturer and the gasoline refiner, and it is increasingly evident that they must work together. Several cases have already been mentioned where improvement of the motor has lightened the demands on the gasoline, and it is probable that many of the problems in unsatisfactory motor performance now laid at the door of the gasoline will be solved by improvement of motor design. The sulfur content of gasoline, for example, is to-day presenting a difficult problem for the petroleum industry because of the high-sulfur crude oils which are being produced in such large quantities. Keeping the sulfur content of gasoline down threatens to cost the petroleum refiner, and of course eventually the public, millions of dollars. If it can be shown that the sulfur bogey can be dispelled by the simple process of ventilating the crank case, a much simpler and easier method of attack will have been found than that which the refiners are following.

The question of anti-knock is another problem being simultaneously attacked from the two different angles by the automobile manufacturer and the petroleum refiner. Considerable progress has been made along both lines and, as G. G. Brown[1] has recently pointed out, the probable outcome will be a moderately anti-knock fuel and a motor which can successfully use this fuel at high compressions.

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[1] *National Petroleum News*, February 22, 1928, p. 87.

DISCUSSION ON GASOLINE PERFORMANCE

Mr. Boyd.

MR. T. A. BOYD[1] (*presented in written form*).—Mr. Hill refers to the fact that "The further improvement of motor performance is in the hands of both the motor car manufacturer and the gasoline refiner, and it is increasingly evident that they must work together." That this is so is a very fortunate thing, both for the two industries concerned and for the public which also is concerned. The active and friendly cooperation that has grown up in recent years, particularly between the technical branches of the two industries, is very gratifying to everyone. And it is only fair to say that in bringing this cooperation about the American Society for Testing Materials has played a most important part.

It now looks as though there will always be need for this close cooperation, because there are some problems that can not be solved satisfactorily by one of the industries alone. Take, for instance, the problem of sulfur in gasoline that Mr. Hill has given as an illustration. It is quite true that engine modifications, such as the ventilation of crank cases and the thermostatic control of jacket water temperatures, are of immense value in eliminating the corrosion trouble that sometimes results from the oxides of sulfur. But, even after these improvements shall have been universally adopted, there will still be need for the refiner to keep the sulfur content of gasoline down to reasonable limits. All of the mechanical devices for preventing trouble from sulfur must and do depend for their operation upon the heat of the engine. But in cold winter weather, when frequent starts and very short runs are sometimes made, the oil and the engine parts cannot always reach temperatures that are high enough to permit these protective devices to function in the normal manner. Consequently they can never remove the necessity for keeping the sulfur in gasoline down to reasonable values. It is not possible now to say just what limit will be placed on sulfur content in gasoline under the improved conditions of the future, but it seems as though the complete solution of the sulfur problem will always demand the help both of the manufacturer of the car and of the refiner of the fuel it uses.

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[1] Research Laboratories, General Motors Corporation, Detroit, Mich.

Mr. Fitch.

MR. T. A. FITCH[1] (*presented in written form*).—This paper is surely an outspoken presentation of gasoline as it is, or should be, to the technically inclined motorist. It disposes of the popular fallacy about great difference in quality among gasolines, and helps relegate to the discard the all-too-common rating of motor fuel by gravity.

The author rightly stipulates that most motorists are not equipped to determine what is a good motor fuel. Yet it is feasible to assure the motoring public of standard quality gasoline. Public regulation and testing, in accordance with the Society's standards, of all gasoline offered for sale has accomplished this in the city of Los Angeles where, at present, gasoline is marketed under more than 75 brands or trade names.

Mr. Abbott.

MR. F. D. ABBOTT.[2]—I wonder if Mr. Hill believes that all these materials put in as an anti-knock material will act in the same way in cutting down this detonation, and if so, in what way do they do that? That is, what is the effect of an anti-knock material in the gasoline-air mixture just previous to or during explosion?

Mr. Hill.

MR. J. B. HILL.[3]—I wish we knew. If we did we would be more successful in developing anti-knock materials. There are as many theories on the action of such materials on gasoline as there are people that are working on it. We do not know how they work; we simply know that they cut down the detonation.

Mr. Abbott.

MR. ABBOTT.—Is there anything in the idea that anti-knock substances slow up the speed of combustion due to intermediate compound formation?

Mr. Hill.

MR. HILL.—If we assume detonation to be a very rapid rate of explosion, anything that cuts down detonation must, of course, cut down the speed of explosion. Just how it does it we do not know.

Mr. Depew.

MR. H. A. DEPEW.[4]—I should like to ask Mr. Hill if on the basis of his paper he would recommend that we buy our gasoline at the cheapest station.

Mr. Hill.

MR. HILL.—It depends, of course, on what you may want. You can probably run your car on gasoline that you buy from the cheapest station, but if you want easy starting, if you want good anti-knock qualities, if you want other things that are probably more or less luxuries in driving a car, I would not do it.

Mr. Delbridge.

MR. T. G. DELBRIDGE.[5]—Supplementing the answer to the last question, I think it would be very unwise to buy gasoline at the

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[1] Testing Engineer, City of Los Angeles, Calif.

[2] Research Physicist, Firestone Tire and Rubber Co., Akron, Ohio.

[3] Chief Chemist, The Atlantic Refining Co., Philadelphia, Pa.

[4] Research Division, New Jersey Zinc Co., Palmerton, Pa.

[5] Process Supervisor, The Atlantic Refining Co., Philadelphia, Pa.

Mr. Delbridge

cheapest station unless you want to dispose of the car rather promptly. One reason is that gasoline at cheap stations is likely to be mixed with kerosine, or something resembling kerosine. If you buy gasoline from a reputable place you can be reasonably sure that the material is gasoline as it was put out by a reputable refiner. If gasoline contains kerosine you will have poor lubrication due to wear which may not reveal itself for months.

Another reason why I should not buy cheap gasoline is because it is not uniform. The ordinary motorist has his carburetor and other accessories to the engine set so that the motor operates satisfactorily for the particular fuel that he prefers, and if you change that fuel you will have different performance, usually less satisfactory. These two reasons alone should send the motorist to a reputable dealer handling a well known and established brand.

Mr. Stillman.

MR. K. W. STILLMAN.[1]—I should like to ask Mr. Hill what he thinks the effect of aluminum cylinder heads will be on detonation. At least one truck maker and one or two car makers have adopted aluminum cylinder heads for the ostensible purpose of eliminating or preventing to some extent detonation with ordinary fuels.

Mr. Hill.

MR. HILL.—That is something that I should much prefer to have a motor car manufacturer answer. My guess is that the principal action of aluminum on detonation is to cut down the sound. The detonation takes place just the same but there is not as much sound.

Mr. Wilson.

MR. R. E. WILSON.[2]—Referring to the use of aluminum pistons, I think that aluminum does in some engines have a definite influence in preventing a hot spot near the center of the piston. However, this does not solve the anti-knock fuel problem. It simply means that the engine builder shoves his compression up a little more to take whatever small share he can of the advantages of higher compression. An engine designed for good efficiency is almost always close to the knocking limit. If carbon forms or the engine gets overheated, it will knock the same as the engine of slightly lower compression which does not have aluminum pistons.

Mr. Faragher.

MR. W. F. FARAGHER[3] (*by letter*).—Mr. Hill has succeeded in presenting a graphic analysis of the significant properties of gasoline that is helpful to the technical man engaged in other fields and even to the non-technical man. His conclusions undoubtedly have in general the approval of technologists directly concerned with gasoline.

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[1] Automotive Industries, Philadelphia, Pa.

[2] Assistant Director of Research, Standard Oil Co. (Indiana), Whiting, Ind.

[3] In Charge of Research and Development, Universal Oil Products Co., Riverside, Ill.

Mr. Faragher.

No attempt is made to show that gasolines for airplanes must meet all the conditions set by the engines of automobiles, and some even more exacting. Perhaps the relatively small amount of gasoline used to-day in aircraft justified this omission. Credible prophets tell us, however, that we shall have another condition by an early "to-morrow."

Mr. Hill rightly accepted without comment gasolines that consist wholly or in part of "cracked" products, indicating that they need conform only to the acceptable values of the selected tests. The usual superior anti-knock properties of these cracked gasolines is simply an added virtue.

Some of the secondary requirements enumerated by Mr. Hill have already been given consideration equal to that of the primary requirements by men who determine what airplane fuel must be. The content of "gum" has been set very stringently by some, and is apparently regarded as a debarring specification. Perhaps the figure that has been chosen is based on adequate investigations. The literature does not, however, disclose this proper basis. The trouble that is caused by "gum" is recognized, and the results are most serious when met in airplanes. It seems to be probable that a proper study of the subject would show that cracked gasolines can be used satisfactorily in airplanes and the selection of suitable cracked gasoline be insured. I believe that consideration of the character of "gum" in a gasoline is as important as consideration of the quantity of it.

Attempts to limit the percentage of unsaturated hydrocarbons in airplane fuels as determined by solubility in sulfuric acid or by effect on the aniline point have already been made. These attempts seem to be ill advised. Certainly the nature of the unsaturated hydrocarbons in a gasoline is of more importance than the total percentage of them.

This discussion is presented solely to point out that since cracked gasoline operates automobile engines properly, it can be expected to operate engines of airplanes satisfactorily. We need a selection of significant tests and properly determined values to insure the safe operation of airplane engines by cracked gasolines. Cooperative tests can be arranged readily.

THE SIGNIFICANCE OF VARIOUS TESTS APPLIED TO MOTOR OILS

By Robert E. Wilson[1] and D. P. Barnard, 4th[2]

Synopsis

This paper considers the relatively large number of testing methods which have been developed for or sometimes used for the testing of motor oils and attempts to evaluate, in the light of all available information, just what significance the results obtained by these tests have as bearing upon the performance of the oil in service.

The tests are grouped under four general heads, as follows:

1. Tests for properties that are important in service, including viscosity and temperature coefficient of viscosity, pour test, and carbon residue;
2. Tests that have little relation to performance but which may indicate contamination or inadequate refining, including color, water and sediment, neutralization number, and corrosion test;
3. Tests that have little or no bearing on performance but which may indicate the probable source of the oil, including gravity, and flash and fire tests;
4. Tests designed for other products but which are sometimes injudiciously applied to motor oils, including those for sulfur content, unsaturation, emulsification, evaporation, precipitation, and analytical data.

The paper also discusses two properties of possible importance, oiliness and resistance to oxidation, for which satisfactory tests are not yet available.

Introduction

Like the makers of firearms and motor cars, Committee D-2 on Petroleum Products and Lubricants of our Society may justly be criticized for creating and supplying to the general public a variety of potent instrumentalities without furnishing the intelligence and judgment necessary to insure that they will be employed with useful, rather than harmful results. Thus, it has standardized twelve tests which, in addition to other unstandardized tests, are frequently applied to motor oils, but until this year the committee has made no serious attempt to guide the average purchasing agent or the general public in their proper practical interpretation and use.

The committee's efforts to remedy this situation have resulted, first, in their preparing a comprehensive report on the scope, significance and applicability of a large number of standard and non-standard methods of testing petroleum products,[1] and, second, in their arranging for two papers to discuss the significance of the various tests applied to motor fuels and motor oils, respectively. In preparing this paper the authors have been aided and guided by the above mentioned report, but their detailed discussion of the relation between test results and performance in service is necessarily based to a considerable extent on their own judgment and experience, as well as upon the more generally known data in the literature.

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[1] Assistant Director of Research, Standard Oil Co. (Indiana), Whiting, Ind.

[2] Director, Engine Laboratory, Standard Oil Co. (Indiana), Whiting, Ind.

The fifteen or more tests which are frequently applied to motor oils (many of which are illogically used in specifications) may conveniently be divided into four general classes as follows:

1. Tests for properties that are important in service;
2. Tests that have little relation to performance but which may indicate contamination or inadequate refining;
3. Tests that have little or no bearing on performance but which may indicate the probable source of the oil; and
4. Tests designed for other products but which are sometimes injudiciously applied to motor oils.

It will probably come as a surprise even to those in the petroleum industry to find that only three tests are placed in the first class, but it is believed that this substantially represents the facts in the light of present knowledge, and that a better recognition of this and other facts set forth in this paper will not only eliminate many unnecessary specifications but will make it more certain that purchasers on specifications will get what they really need.

In addition to these three tests there are at least two properties of possible importance for which no satisfactory standardized tests are available.

TESTS FOR PROPERTIES IMPORTANT IN SERVICE

Viscosity and Temperature Coefficient of Viscosity:

Viscosity is undoubtedly the most important single characteristic of any lubricating oil from the point of view of performance. As has been previously pointed out by the authors,[2] the viscosity of an oil is the property that determines the friction in any bearing or between any two surfaces separated by a fluid film of oil, and also largely determines the load which can be carried under given operating conditions before film rupture takes place. In selecting an oil, the first requisite is to choose one of high enough viscosity to insure the presence of a fluid film between all moving surfaces under all normal conditions of operation, plus a factor of safety, and yet not to choose one so high in viscosity that it causes excessive friction, heating, hard starting, or poor circulation.

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[1] See Appendix to Report of Committee D-2 on Petroleum Products and Lubricants, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 479 (1928).

[2] *Journal*, Soc. Automotive Engrs., Vol. XI, p. 49 (1922).

For machines used at a roughly constant temperature, there is usually a fairly wide viscosity range between the lower limit where there is danger of film rupture and the upper limit where power consumption becomes excessive; but in an automobile which must be used over a wide range of temperature conditions careful selection is necessary to secure an oil which will give reasonably easy starting and proper circulation at temperatures around zero and yet have adequate viscosity or body at the higher operating temperatures such as prevail after the engine warms up in a hard run. Such temperatures frequently range as high as 175° F. in the crank case even in winter, and still higher temperatures may prevail in the bearings and on the cylinder walls.

Fortunately, the measurement of viscosity by means of the Saybolt Universal Viscosimeter[1] is a rapid and accurate test which is almost universally recognized throughout the country. Since oil viscosities change rapidly with temperature, and at different rates, it is desirable and customary to measure the viscosity at a temperature approximating average operating conditions, and for this reason the viscosity of the lighter grades of motor oils is generally measured at 100 or 130° F. and that of the heavier grades at 210° F. The use of higher temperatures for the heavy oils also cuts down the time required for a test.

While the Saybolt readings (time of efflux in seconds) can readily be converted into absolute viscosity,[2] it must be remembered that the number of seconds ordinarily reported as the viscosity of a motor oil is not directly proportional to its absolute viscosity.

Not only is it important to have an oil of the correct viscosity for a given car under average operating conditions, but it is also highly desirable to have an oil whose viscosity changes as little as possible with change in temperature. It has, of course, long been known that two oils may have the same viscosity at 210° F. and a distinctly different viscosity at 100° F.; for example, two typical heavy motor oils from different crude oils with the same viscosity at 210° F. (55 seconds) have a viscosity of 525 and 420 seconds respectively at 100° F. While this is a marked difference, it was not until viscosity measurements and cranking tests were recently made at 0° F. and below[1] that the full effect and magnitude of this difference in trend was appreciated, for these same two oils show viscosities at 32° F. of 15,000 and 7,300 seconds, respectively, and at 0° F. of 280,000 and 66,000 seconds. (Of course such determinations cannot be made in the Saybolt instrument as conventionally used, but can be calculated from measurements made in a slightly modified instrument at higher pressures.)

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[1] A.S.T.M. Standard Methods of Test for Viscosity of Petroleum Products and Lubricants (D 88 – 26), 1927 Book of A.S.T.M. Standards, Part II, p. 427.

[2] *International Critical Tables*, Vol. I, p. 32, 1926 Edition.

This work also showed that the effort required to crank an engine at a given speed depends solely upon the viscosity of the motor oil at the prevailing temperature, so that the difference between the two oils which seem moderate at 100° F. becomes extremely important at lower temperatures and may mean the difference between satisfactory and impossible cranking at a given temperature.

The increasing appreciation of the importance of the temperature coefficient of viscosity raises a question as to the best method of determining and specifying it. Where any attention has been paid to this point, it has been done by specifying the viscosity at two different temperatures, usually 210° F. and 100° F. This method of limiting the viscosity coefficient is, however, not entirely satisfactory, for unless the viscosity ranges at each temperature are made unreasonably narrow, an oil which is near the low limit at 210° F. and near the high limit at 100° F. might have a very much higher viscosity at zero than an oil which approaches the opposite limits. From a theoretical standpoint it would be preferable to specify separately the two properties of viscosity and temperature coefficient, the latter possibly being expressed as the ratio of the viscosity at 100° F. to that at 210° F. Of course, the precise ratio obtainable from a given type of crude oil varies with the viscosity of the cut, but it is believed that some such specification is important if one is to insure satisfactory performance under a wide range of temperature conditions.

In specifying the viscosity of motor oil, the factor of dilution must also be taken into account. Except for the equilibrium or prediluted type of oil, the viscosity of the oil put into the crank case of a car will, under average winter operating conditions, drop to one-half or one-third of its original value in about 200 miles of operation due to its absorbing 10 to 15 per cent of its weight of the heavy ends of gasoline. This means that either the fresh oil must be heavier than the optimum, or the oil after 200 miles will be too thin for the best results. In summer operation the effect is, of course, much smaller, though still significant.

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[1] *Journal*, Soc. Automotive Engrs., Vol. XXII, No. 2, p. 213 (1927).

Pour Test:

The A.S.T.M. Tentative Method of Test for Cloud and Pour Points of Petroleum Products (D 97 - 27 T)[1] determines the lowest temperature at which an oil will flow in a small jar when tilted under specified conditions. The tendency of many oils to solidify at lower temperatures is generally due to the presence of small quantities of paraffin wax which stay in solution at the higher temperatures but separate out as lower temperatures are reached.

The pour test undoubtedly measures the second most important property of motor oils so far as winter operation is concerned. While the above-mentioned paper[2] showed that cranking effort depends on viscosity alone and does not change appreciably with variations in pour test, it also showed that oil circulation in many engines, particularly those in which fine oil screens are used, does not begin at temperatures below the pour point of the oil in the crank case. The pour test is also a fairly accurate measure of the temperature below which the oil cannot readily be poured out of a container such as a five-gallon can. It is, therefore, of real importance to have a low pour test in an automobile lubricant, even though it has no direct bearing on the ease of starting or on the ability of the oil to flow under high pressures such as those prevailing in the pressure side of the lubricating system.

Carbon Residue:

The A.S.T.M. Tentative Method of Test for Carbon Residue of Petroleum Products (D 189 - 27 T)[3] determines the amount of coke-like material left behind when an oil is subjected to destructive distillation under specified conditions at atmospheric pressure. While the significance of this test has been the subject of considerable dispute, recent evidence[4, 5] indicates rather clearly that when using the same engine under carefully controlled conditions the carbon deposit in the combustion space is a function of this carbon residue value of the oil, though the carbon deposit increases much less rapidly than in direct proportion to this value. Of course, if the oils have different viscosities at the operating temperatures the one with lower viscosity tends to pass the rings in larger quantities and thus affect the amount of carbon formation. Furthermore, in actual service the condition of the engine, and particularly the type and fit of the rings, as well as changes in mixture ratio, operating temperature, oil viscosity, etc., will obscure or outweigh ordinary differences in carbon residue between two different oils. In spite of this, however, all other things being equal, an oil of lower carbon residue is less likely to give carbon troubles than an oil of similar viscosity and distinctly higher carbon residue. Of course, improvements in engine design and workmanship are tending to decrease oil consumption and minimize carbon formation.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 919 (1927); also 1927 Book of A.S.T.M Tentative Standards, p. 390.

[2] *Journal*, Soc. Automotive Engrs., Vol. XXII, No. 2, p. 213 (1927).

[3] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 915 (1927); also 1927 Book of A.S.T.M Tentative Standards, p. 386.

[4] *Industrial and Engineering Chemistry*, Vol. 17, p. 502 (1926).

[5] *Journal*, Soc. Automotive Engrs., Vol. XVIII, p. 48 (1926).

TESTS THAT HAVE LITTLE RELATION TO PERFORMANCE BUT WHICH MAY INDICATE CONTAMINATION OR INADEQUATE REFINING

In so far as the tests in this class may be used to eliminate poorly refined or contaminated oils, there can be no sound objection to their inclusion in specifications, but it should be realized that they do not go to the heart of the problem and that overly strict requirements as to properties such as color or organic acidity, are not justified.

Color:

The A.S.T.M. Tentative Method of Test for Color of Lubricating Oils by Means of Union Colorimeter (D 155 – 23 T)[1] employs the Union colorimeter for measuring the color of a lubricating oil as it appears in a 4-oz. bottle. While it is generally true that the darker the oil the higher is likely to be its viscosity and carbon residue, there are so many exceptions to this generalization that it is absurd to specify color as an indication of properties such as these which can be measured directly. Its main value as a test is in checking the uniformity of a given brand of oil or in detecting contamination by dark colored products. In comparing oils of the same viscosity made from a given crude oil, it is generally true that the more complete the refining the lighter the color. This, however, does not necessarily mean any better performance in an internal combustion engine, and the test is of absolutely no value in judging the relative degree of refining in oils made from different crude oils.

Water and Sediment:

Water and sediment are never found in an oil except when the product has become contaminated in handling and ordinarily no quantitative tests are required, their presence being apparent on inspection.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part I, p. 677 (1923); also 1927 Book of A.S.T.M. Tentative Standards, p. 397.

Neutralization Number:

The A.S.T.M. Tentative Method of Test for Neutralization Number of Petroleum Products and Lubricants (D 188 - 27 T)[1] may be used to determine either (1) the approximate quantity of organic material having acidic characteristics, or (2) the water-soluble free alkali or mineral acid which may be present in a lubricating oil. The presence of an appreciable quantity of alkali or mineral acid is an indication that these refining chemicals have not been adequately removed from the oil, but this condition is seldom found. While the method is also frequently used to determine small amounts of organic acids which are likely to be present in oils made from certain crude oils, there is no evidence that such small quantities of extremely weak oil-soluble acids are in any way harmful in service, at least in the quantities ordinarily encountered. In fact, there is definite evidence that even larger quantities of free fatty acid are harmless in service.

Corrosion Test:

The Federal Specifications Board has adopted Method 530.31 which determines the corrosive properties of lubricating oil by heating a copper strip therein for 3 hours at 212° F. Such a test would probably indicate the presence of either free mineral acids, which may be better determined by the neutralization number method just discussed, or certain types of sulfur compounds which are not at all likely to be present in lubricating oils and which probably would not be harmful to the metals present in the crank case. The authors have never seen a case where the oil itself (as distinct from water which might condense therein) has definitely caused corrosion in the cylinders or crank case. The method has, therefore, not yet demonstrated its value as a test for motor oils.

TESTS THAT HAVE LITTLE OR NO BEARING ON PERFORMANCE BUT WHICH MAY INDICATE THE PROBABLE SOURCE OF THE OIL

The use of "source identifying" characteristics as a basis for specification is largely an outgrowth of competitive selling which urged that the origin of the crude oil, rather than the character of the oil itself, was the best indication of quality. While this may have been excusable before satisfactory tests were available for the really important properties, improvements in testing methods and in the technique of refining various crude oils have left no excuse for the use of such tests as a basis for specifications in place of the properties actually desired. Furthermore, arbitrary requirements of non-essential characteristics always tend to hamper the development of new and better products.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part I, p. 927 (1927); also 1927 Book of A.S.T.M. Tentative Standards, p. 418.

Gravity:

The specific or A.P.I. gravity of an oil is an excellent example of the above mentioned type of property. It is probably true that if one were to make a survey of all the motor oils on the market of a given viscosity range, the oils which had relatively high A.P.I. gravities would on the average be found to have better temperature-viscosity coefficients, higher pour tests, and higher carbon residues than the oils with lower A.P.I. gravities. On the other hand such a survey would show so many marked exceptions to this generalization as clearly to demonstrate the fallacy of using as a basis for specifications such a property which has no direct importance.

Flash and Fire Tests:

While the flash test is an important test for products such as kerosine, in that it indicates the safety with which it may be handled, the flash test of all lubricating oils is so high that the safety factor is of no importance whatever. Furthermore, a typical motor oil in average service loses about 200° F. in flash point in the first 100 miles of operation, due to crank case dilution. The flash test is also sometimes considered as a measure of the volatility of an oil but it really measures the evaporating tendency of the first few per cent of an oil, which certainly has no practical importance. The real reason for the retention of a flash-test requirement in many motor oil specifications is the fact that, when taken in connection with other properties, it is to some extent a source-identifying characteristic. As such it is open to the objections discussed above.

TESTS DESIGNED FOR OTHER PRODUCTS WHICH ARE SOMETIMES INJUDICIOUSLY APPLIED TO MOTOR OILS

Sulfur Content:

While the sulfur content of gasoline is important because the gasoline is burned and part of the sulfur forms sulfuric acid, which may cause corrosion of the working parts in winter operation when water is likely to condense on the cylinder walls and in the crank case, there seems to be no evidence whatever that the sulfur content of a motor oil has any bearing on its behavior in service. Since, in general, less than 1 per cent as much oil as gasoline is actually burned, the sulfur content of the oil can have no appreciable bearing on the amount of sulfuric acid formed or the amount of corrosion which might take place. Organic sulfur compounds of the type likely to be found in lubricating oils are just as stable under crank case conditions as the hydrocarbons themselves.

Unsaturation:

While the bromine numbers or other supposed measures of the amount of unsaturated hydrocarbons present may rightfully be included in the specifications for certain petroleum products for special purposes, they seem to have no bearing whatever on the performance or stability of motor oils and hence no rightful place in motor oil specifications. Furthermore, most tests of this character give variable and meaningless results when applied to products as heavy as lubricating oils.

Emulsion Tests:

Standardized tests to determine the ease of emulsification or the difficulty of breaking emulsions are important in the case of turbine oils and other oils which regularly come in contact with large quantities of water. In the case of motor oils, such tests seem to be of negligible importance, because the amount of water which gets into a crank case is generally small and it is a matter of dispute as to whether it is better to have this water separate out readily on the bottom of the crank case or remain emulsified with a better chance to be evaporated. It is true that oils containing substantial quantities of soaps which tend to give stiff emulsions with small amounts of water are likely to give trouble in winter service, but such oils are not often employed and can be detected by simply shaking with a little water, without necessitating a strict specification based upon the precise rate of separation of the water from the oil in the standard emulsion tests, especially since this changes very rapidly after the oil is put into the crank case. Frequently oils with poor initial emulsion tests will deteriorate in this respect less rapidly in service than oils with good initial tests.

Evaporation Test:

The tendency toward higher operating speeds which cause high oil consumption has raised the question as to whether some of the oil may be evaporated at high crank case temperatures, and tests have been proposed to measure the rate of evaporation of the light ends of the oil. In view, however, of the fact that even the higher operating temperatures are several hundred degrees below the initial boiling point of the oil and that under practically all conditions the crank case oil tends to condense and dissolve some of the heavy ends of gasoline instead of vaporizing the more volatile part of the oil; the need for such a test for motor oils does not seem to be apparent except possibly for extreme conditions of service, such as racing. It is certain that if an oil is heated to a temperature high enough to vaporize noticeably, it has lost most of its viscosity and an engine should be designed to prevent the attainment of such excessive temperatures, which are necessarily harmful in other respects as well. Evaporation tests are sometimes supposed to throw light on the cracking tendency of an oil, but the temperatures employed are far too low for this purpose.

Precipitation Test:

The A.S.T.M. Tentative Method of Test for Precipitation Number of Lubricating Oils (D 91 - 21 T)[1] determines the amount of material, generally of an asphaltic nature, that is insoluble in petroleum naphtha of specified properties. While it is useful in testing black oils and unrefined cylinder stocks, refined motor oils never contain such asphaltic material and the test is unnecessary.

Analytical Data:

Specifications based on chemical analysis may, of course, be properly used to insure the absence or presence in definite quantity of soaps, fats, fatty acids, etc., which are mainly employed in oils made especially to lubricate the transmission bands of a car now obsolete. For the ordinary motor oils which are blends of pure petroleum products, analytical data have little or no significance.

PROPERTIES OF POSSIBLE IMPORTANCE FOR WHICH NO SATISFACTORY TESTS ARE YET AVAILABLE

Since this paper has pointed out a great many tests which are of little or no significance in measuring the value of a motor oil, it may seem surprising that there are at least two properties of possible importance for which no satisfactory tests are yet available. This may be explained largely on the basis that the tendency is to first develop tests for properties which are easy to measure without much regard to their significance, whereas properties such as oiliness and resistance to oxidation are not only extremely difficult to measure but very hard to correlate with actual performance in service. The comments on these two subjects are therefore necessarily more or less tentative.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 21 p. 648 (1921); also 1927 Book of A.S.T.M. Tentative Standards, p. 427.

Resistance to Oxidation:

After extensive work covering a period of many years and involving comparative tests with several different types of methods, Committee D-2 on Petroleum Products and Lubricants has developed a method known as the Sligh oxidation test which is probably the best now available for measuring the rate of oxidation of an oil under controlled conditions. Unfortunately, however, it has not yet been found possible to obtain good checks between different laboratories nor has it been proved that the results thus obtained bear any direct relation to performance in service. Since the oxidation of motor oils takes different courses under different conditions and an oil which is better under one set of conditions may be much poorer under another, it is dangerous to adopt such a test until its relation to service conditions is directly proved. Some laboratories consider that the best available measure of oxidation is to note the change in color of an oil when kept in contact with air at somewhat elevated temperatures. It is believed by many that the darkening of motor oils in service is due to oxidation and some also claim that the formation of a deposit on the connecting rods, etc., and the gumming of the rings, which may take place in heavy duty engines such as airplane engines, is due to oxidation. Again this has not been definitely established. Probably the most important line of investigation which Committee D-2 should pursue during the next year or two is to try to clear up this question and the related one of the effect of the gum content of gasoline.

Oiliness:

While it has been definitely proved that there is no appreciable difference between the performance of animal and vegetable oils as compared with straight mineral oils of the same viscosity in the ordinary operation of bearings or other well-lubricated surfaces, it has also been established that there are differences between such oils in their ability to adhere to surfaces and lower the friction under very severe conditions of heavy loads and slow speeds.[1] The differences between the behavior of such oils under these abnormal conditions is considered to be an indication of the "oiliness" of the oil and is undoubtedly dependent upon its ability to be adsorbed by metal surfaces. With the exception of the lubrication of the bands of a planetary transmission, it has not been definitely demonstrated that oil of high "oiliness" is of any real advantage in the operation of an automobile engine, and unfortunately practically all of the materials which might be used to increase the "oiliness" of a straight mineral oil have other serious drawbacks to their use—particularly the tendency to form stiff emulsions or to separate out sludgy material in service. Partly for this reason and partly because of the inherent difficulties in measuring quantitatively the rather elusive properties of "oiliness," there is no satisfactory test for reproducibly measuring moderate differences, though several methods have been devised by the authors and others[1] which are adequate to show up large differences between different oils. Such tests would be very hard to standardize as they necessitate accurate reproduction of the surface condition of metals, etc.

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[1] *Journal*, Soc. Automotive Engrs., Vol. XI, p. 143 (1922), Vol. XVII, p. 287 (1925), and Vol. XIX, p. 356 (1926).

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[1] *Journal*, Soc. Automotive Engrs., Vol. XI, p. 143 (1922), Vol. XVII, p. 287 (1925), and Vol. XIX, p. 356 (1926).

DISCUSSION ON TESTS APPLIED TO MOTOR OILS

Mr. Lederer. MR. E. R. LEDERER[1] (*presented in written form*).—Referring to the paper by R. E. Wilson and D. P. Barnard, I should like to call attention to the need for the standardization of an apparatus and procedure to determine fluidity or viscosity at low temperatures. Messrs. Wilson and Barnard refer to such an instrument as a modification of the Saybolt viscosimeter.

The fluidity test of motor oils is significant since this property is very important in service. As pointed out in their paper it determines the effort required to crank an engine in cold weather, and consequently has direct influence on crank case dilution and to the use of the choke in starting. Mr. Barnard described his apparatus in the paper presented at the annual meeting of the Society of Automotive Engineers, held in January of this year. Several of the Government laboratories and many of the oil companies have similar fluidity or low temperature viscosity machines to study the behavior of motor oils at temperatures below their pour test under pressures maintained by an oil pump.

Since this type of apparatus is being used so widely and one that will give satisfactory check results can be constructed at a comparatively low cost, some steps toward standardization should be taken.

Mr. Mougey. MR. H. C. MOUGEY.[2]—I feel that this paper by Wilson and Barnard is very timely and constructive. This paper and the report on Significance of Tests of Petroleum Products presented by Committee D-2 on Petroleum Products and Lubricants appear to mark a turning point in the relations between the oil industry and the consuming public. In the past, there was a lot of mystery in oil tests. However, in the future, with a more definite understanding of the significance of these tests, rapid progress should be made in obtaining better lubricants more suited to the individual requirements of the consumer.

Examples of this are seen in the recent developments of the oil refiners. Low pour test oils having a low temperature coefficient of

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[1] Vice-President and Director, Texas Pacific Coal and Oil Co., Fort Worth, Tex.

[2] Chief Chemist, General Motors Corporation, Research Laboratories, Detroit, Mich.

Mr. Mougey. viscosity are now being produced by several of the large refiners. Oil of low volatility and high viscosity, designed for high-speed, high-temperature service, are now being made from practically all types of crude oil. Recent developments have also shown that oils with low Conradson carbon residue values can be refined from all types of crude oil. These developments were possible only through a better understanding of the significance of oil tests.

The developments resulting from the proper interpretation of these tests are also of great value to the automobile engineers, as it is their aim to design engines which will operate satisfactorily with oils from all crude oils.

A number of tests have been pointed out as having little or no value for determining the suitability of an oil for a particular type of service. It should, however, be emphasized that these tests are of great importance in identifying oils, checking operations in the refinery, and in checking shipments to determine if they meet the commercial requirements. By a better understanding of these tests, improper use of these tests for either service or commercial reasons will be decreased.

Mr. Herschel. MR. W. H. HERSCHEL.[1]—It is very gratifying that Committee D-2 has undertaken to describe the significance of tests concerning which a considerable amount of misinformation has been published. Following is an example:

> "Viscosity measures fluidity, and fluidity alone, and the application of the word viscosity to fluidity by the oil trade was charlatanism, pure and simple, even though in all fairness to Professor Saybolt, the inventor of the term viscosity, it may be truly said that he had no such intention."

I am quite prepared to agree with Wilson and Barnard that color, flash point and various other tests in common use are of little importance compared with the three tests classed by them as important in service. On the other hand the temperature coefficient of viscosity has not received the attention it deserves. A specification of viscosity at a single temperature may be entirely inadequate, as when an oil meets the viscosity specification at the test temperature but will not run out of the barrel at room temperature.

It is not clear what is meant by the title of the second group of tests. The only object of refining is to make an oil that will give adequate performance, and thus if the performance is adequate, so is the refining.

The need or importance of a test depends upon what other tests are included or omitted. Flash point is a crude evaporation test,

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[1] Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

Mr. Herschel. and the neutralization number is less important if the more significant emulsion and oxidation tests are used. The emulsion test shows what an oil is like when new, while the oxidation test indicates what it will become, which is more important. However, since a sludge is an emulsion of water and oxidized oil, both tests are desirable.

One might question the statement that organic sulfur compounds are just as stable as the hydrocarbons themselves. Waters[1] found a very decided increase in oxidation as the sulfur content increases. Thus on account of the effects of sulfur on sludging and on crank case corrosion, a sulfur test appears necessary, unless oils high in sulfur are eliminated by the oxidation test.

Even the most widely accepted tests often prove unsatisfactory with time and require revision. The real question concerning the oxidation and oiliness tests is not whether they are satisfactory but whether these tests, as thus far developed, give information which cannot be obtained by other means. I believe that this question may be answered in the affirmative.

The authors, in view of the valuable pioneer work which they have done, are too modest in regard to the test for oiliness. We have found it possible, by a modification of the disk friction machine used and abandoned by them, to get reliable measurements of oiliness, and find very definite variations in the coefficient of friction depending upon the lubricant and the bearing metal, under conditions such that speed, temperature and viscosity have no effect within the range investigated. With a petroleum oil, the friction decreases from about 0.16 to 0.10 as the percentage of lead in the bearing metal increases. If there is little or no lead in the bearing metal, the addition of a fatty acid to the petroleum oil has little effect, but the benefit to be derived from the use of a fatty acid increases as the percentage of lead in the bearing metal increases. The lowest coefficient of friction, 0.02, was obtained with lard oil on a bearing metal containing 95 per cent of lead. Most fatty oils are too acid and are corrosive on metals high in copper, so that the friction is increased. The friction is decreased by an amount depending upon the lead content of the bearing metal, when a fatty oil low enough in acidity is used. Wesson oil, for example, gives good results, but some of the dopes on the market increase the friction instead of decreasing it.

It is surprising that compounded motor oils, or oils containing fatty acids, should be extensively used in England but not in this country. The authors state that "it has not been definitely demonstrated that oil of high oiliness is of any real advantage in the operation of an automobile engine." This is contrary to the conclusions of Stanton,[1] who has presented evidence that the conditions under which oiliness is of advantage do obtain in the cylinders of automobile engines. **Mr. Herschel.**

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[1] *Industrial and Engineering Chemistry*, Vol. 14, p. 725 (1922).

Mr. Flowers. MR. A. E. FLOWERS.[2]—Messrs. Wilson and Barnard have laid a good deal of stress on the viscosity temperature coefficient. While this may be of considerable importance in the case of motor oil in automobiles, because of the great range of temperatures between winter and summer and between starting and running, in other services where the temperatures do not vary so much, this viscosity temperature coefficient may not only not be a disadvantage but may even be a considerable help. With the increase of temperature and the consequent decrease in viscosity, there is necessarily a decrease in the total amount of friction, and therefore, in the amount of heat produced. This tends to limit the further rise of temperature by decreasing friction and the heat produced thereby.

The other point I wish to bring to your attention is that in the case of the paraffin-base oils under low temperature, what one is primarily concerned with is not the viscosity in that range but the conditions of plastic flow. The authors have referred to it only in terms of increase of pressure for causing flow, but what should have been added to that is the fact that under the conditions of plastic flow one has two terms for the resistance to flow, one of them quite independent of the velocity of motion and the other depending upon a coefficient times the first power of the velocity; that is, the friction force is expressed by the formula $F = A + BV$.

In regard to the question of oiliness, which has been brought out as possibly of great interest but not yet in such shape as to allow any specific tests to be applied to it, the American Society of Mechanical Engineers, through its Research Sub-Committee on Lubrication, is proposing to study this factor of "oiliness" and has prepared a considerable program of the work which it would like to do and for which some moneys will be required. The Research Committee of the A.S.M.E. is attempting now to obtain contributions to a fund which it hopes may reach $10,000, with at least $5000 made available in the next year, in order to make it possible to carry forward this research program.

The reason for wishing to proceed at once at this particular time is that it just happens that previous groundwork for that research has been rather completely laid. The personnel that has been trained in the methods of that study and in the development of instruments

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[1] Aeronautical Research Committee, Reports and Memoranda No. 931 (1925).

[2] Engineer, In Charge of Development, The De Laval Separator Co., Poughkeepsie, N. Y.

Mr. Flowers. for its measurement is available now and may not be available if the program is delayed for even one year. The committee has included tests with at least three distinct types of instrument especially developed for the purpose of studying this oiliness factor. Most of this work has been done at the U. S. Bureau of Standards through the efforts of Mr. M. D. Hersey, in charge of that Section, and with Mr. W. H. Herschel and the other members of the Bureau.

Mr. Parsons. MR. L. W. PARSONS.[1]—It is a pleasure to acknowledge the type of paper presented by Messrs. Wilson and Barnard. The critical surveys of the authors published some time ago on corrosion and on fuel volatility have now been extended to the realm of motor oils. The reference to work on fluidity of motor oils indicates clearly the importance of further work on both fluidity and pour point. Results obtained several years ago in our laboratories, work of the U. S. Bureau of Standards, the reported results of Mr. Lederer and others, indicate the type of equipment involved and the actual fluidity characteristics of various oils under conditions approximating commercial practice. Peculiarities in fluidity and particularly in pour point of several oils have been explained in certain cases by an examination of the microstructure of the oils at low temperatures.

A very definite economic problem is involved when demands are made for very low pour point paraffin-base oils. The cost of refinery equipment, particularly refrigeration, is very large and it is important to establish definitely the need for a low pour point before requiring either the oil refiner or the automobile manufacturer to include a rigid pour point specification. Recent satisfactory long distance airplane flights have been made using both high pour point and low pour point oils, and although for certain types of airplane work there is an artificial heating device available to heat the oil before it is poured into the engine, one cannot limit the range of applicability of either the low or high pour point types of oils.

Most of us are in agreement with the authors regarding the group of tests which have little or no relation to actual performance. Many of these are control tests of value within a given plant to serve as a guide in refining operations. An examination of used oils, even after a very few minutes running in an automobile engine, shows the futility of stressing too rigidly such specifications as color and demulsibility in motor oils, unless a clear understanding of the physical properties and stability of the oils is known.

With reference to measurements on oiliness, I believe that more emphasis is needed on the measurement of oiliness at high tempera-

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[1] Technologist, Tide Water Oil Co., Bayonne, N. J.

Mr. Parsons.

tures, nearer actual operating conditions. Work along these lines has been conducted by the Westinghouse laboratories. In connection with evaporation tests it is possible that the pendulum may swing in the future toward a more careful analysis of motor oils from the standpoint of their boiling point ranges. Here again the value of some class of tests must be determined and clearly understood before being used as an additional specification for the oil refiner to meet.

The importance of emphasizing not only the source of a motor oil but the care taken in refining the crude oil to produce a finished lubricant has been clearly indicated by the authors, and the trend away from many questionable tests, such as gravity, for classifying various oils regarding actual performance, has been definitely recognized.

It is desirable to emphasize strongly in connection with the papers presented on both gasoline and motor oil the feelings of reputable oil companies regarding the use of inferior fuels and lubricants for automotive engines. We believe that the motorist is entitled to a quality of gasoline and lubricant sufficiently high to permit not only operation of his engine, but also a definite factor of safety during such operation. While there are undoubtedly cases where a very cheap oil or gasoline will work for a period of time in certain cars, there is no guarantee that such products will function satisfactorily for the millions of cars on the road to-day and during the extended period they are operating. I do not believe there is any one thing that those of us who are in the oil refining business are more interested in or more jealous of than our trade name. Back of an oil company's product lies all the painstaking work in selection of crude oils, methods of refining, control during processing, experimental and actual road testing, and fixing of specifications, which finally culminates in our advertising under well-established company branded names. Practically all of the oil companies to-day maintain definite research organizations to study the characteristics of different types of automobile engines and to develop real quality fuels and lubricants which will permit operation with a definite factor of safety.

Mr. Delbridge.

MR. T. G. DELBRIDGE.[1]—The subject of motor oil, like that of gasoline, is one of direct personal interest to most of us, and it may not be amiss to point out a possible pitfall for the user of motor oil.

The field of gasoline "dopes" has been a very fertile one, and with one or two exceptions there are no dopes that are worth anything in motor fuel. Apparently we are now going to have "dopes" for motor oils, and there are being offered to the public at the price

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[1] Process Supervisor, The Atlantic Refining Co., Philadelphia, Pa.

Mr. Delbridge. of a dollar or more for 8 ounces, or less, certain oils which if added to your usual brand of motor oil will halve your repair bills and do many other things. One of these is a few hundredths of one per cent of rubber added to motor oil. Oiliness, that is, the character of the oil in a thin film, is a very obscure matter, and, therefore, it is going to be very simple for the enterprising advertiser to put across over the motoring public the same kind of campaign that has been put over on many of us in connection with gasoline dopes.

Mr. Herschel. MR. HERSCHEL.—In regard to "dopes," we have examined with our friction machine some of the rubber cement which Mr. Delbridge mentioned, and we find it increases the friction. On the other hand, there are dopes on the market which will decrease the friction. One of them costs only 55 cents a gallon, and it is just as good as the Wesson oil which I previously mentioned.

To change the subject, I believe that both Mr. Flowers, and Mr. Wilson in presenting the paper, mentioned the equation of flow, that the flow was proportional to the pressure to the *n*th power. Unfortunately, the matter is not quite so simple as that. When one gets to very stiff materials one has a more complicated equation of flow; the flow is proportional to $(\text{pressure} - \text{constant})^n$ and I believe if you had an exponent as high as 2—as used by Mr. Wilson to calculate a curve showing the change of velocity on the diameter of a pipe—you would get that more complicated form of equation. I should, therefore, like to ask Mr. Wilson where he gets that form of change of velocity on the diameter.

Mr. Fulweiler. THE CHAIRMAN (*Past-President W. H. Fulweiler*[1]).—If the meeting will permit and before I ask Mr. Wilson to close the discussion, I should like to call attention to an observation that we have recently made on the effect of oxidation and sulfur compounds.

For a number of years we have been unable to explain why certain unsaturated hydrocarbons oxidize very rapidly in the presence of low concentrations of oxygen, say of the order of 0.4 to 0.8 per cent. The pure hydrocarbons did not show this rapid oxidation. We have recently found that by adding a small quantity, say of the order of 0.004 to 0.01 per cent of a mercaptan, say phenyl mercaptan, the rate of oxidation increased several thousand times.

While this observation may not directly confirm Mr. Herschel's suggestions that with high sulfur oils we will always have high oxidation yet this is one case at least that with a normal temperature and low concentration of oxygen the presence of a very small amount of sulfur compounds increased the oxidation rate very greatly.

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[1] Chemical Engineer, United Gas Improvement Co., Philadelphia, Pa.

Mr. Wilson.

Mr. R. E. Wilson.[1]—With regard to the point Mr. Herschel makes about adequate refining, I think it is clear to most refiners what we have in mind. I should point out, in the first place, that I did not say these tests had no relation to performance, but that they had little or no such relation. Take, for example, the question of water and sediment, I do not believe that 1 per cent of water left in motor oil is going to do any appreciable harm under ordinary conditions, certainly not in summer, or that a little trace of alkali left in the oil after neutralization is going to do any harm. It is, however, poor refining, and any refiner who takes any pride in his products is not going to leave in materials of this type which can readily be removed. While we cannot say definitely that they are likely to do any particular harm, it is not good refining to leave them in.

With reference to the point Mr. Herschel made with regard to the sulfur content possibly having some influence on oxidation, which is not improbable, when I spoke about the stability of the sulfur compounds in motor oil being as high as that of the hydrocarbons, I of course had in mind stability against thermal decomposition. If there is some obscure or rough general relationship between sulfur content and oxidation, I would ten times rather measure oxidation, which is what we are interested in, and not sulfur content. In other words, let us measure the property we really want and not some indirect possible indication thereof.

I think the same is true of the emulsion test. Of course, as Mr. Herschel points out, the emulsion test changes rapidly with time in the engine, so that the initial emulsion test does not mean much, and while it is possible that an emulsion test might give some indication of the sludging, I think that that by no means is the only thing which it will give. For example, while Mr. Herschel spoke about the emulsion test measuring acid formation due to oxidation, a tiny bit of soap from one source or another will have a tremendous influence on the emulsion test, yet it would have nothing to do with oxidation.

With regard to Mr. Flower's comment on plastic flow laws, I should like to ask if he was referring to Bingham and Green's old equation for yield point? that is, where they divide up the flow pressure diagram into yield point and mobility?

Mr. Flowers.

Mr. Flowers.—Yes.

Mr. Wilson.

Mr. Wilson.—The point is that while that is fairly well established as holding true for paints and other cases where one has essentially a suspension, the flow of materials like starch paste, or like these oils, does not follow that law. One cannot get even an approximation.

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[1] Assistant Director of Research, Standard Oil Co. (Indiana), Whiting, Ind.

Mr. Wilson. That is, the curvature does not fit into that type of approximation. The curvature is extremely gradual and apparently keeps on curving just as far up as one can measure, and, in answer to Mr. Herschel's question, over quite a range, it is a simple exponential formula. Most of our experimental values were below 2, but we did find that similar high exponential values held with good accuracy over a twenty-fold range of pressure variation. With our instrument, which is simply a means for putting high pressure and maintaining low temperature around an ordinary viscosimeter, we get a straight line on log paper, and therefore an exponential relationship between pressure and flow. The curve I showed in presenting the paper, for flow in a pipe is simply calculated from the laws of flow on the assumption that one has an oil whose exponent is 2. We could make a similar calculation for one whose coefficient is say 1.6, a fairly typical figure, and would get a flow diagram of intermediate character. The curve shown was intended merely to show in a simple and qualitative manner the radically different character of flow conditions for the plastic type of oils.

STAINING AND EFFLORESCENCE ON INDIANA LIMESTONE CAUSED BY MOISTURE SEEPAGE THROUGH BACKING MASONRY MATERIALS

By Lee Huber[1]

Synopsis

This paper contains descriptions of the source, cause, mechanism and prevention of efflorescence and staining on Indiana limestone caused by moisture seepage through the wall backing material. Most of the statements made are substantiated by actual observation or experimental work. This is especially true of the section dealing with staining.

The results obtained indicated very strongly that the best method for the prevention of both efflorescence and staining lies in the following of proper construction practices which eliminates all moisture seepage through the wall.

Experiments on twenty-one different brands of commercial waterproofing materials show that they could not be depended upon for preventing the occurrence of efflorescence or staining.

An experiment performed by parging the back of stone with a mortar made from a good waterproofed cement indicated that this is a very promising line of attack for the prevention of efflorescence or for the prevention of any other phenomenon the existence of which is maintained by the seepage of moisture through the wall.

There are probably no two things that cause the stone industry more trouble than the staining and efflorescence which quite commonly occurs on stonework where the construction conditions are not just right. With this fact in mind the former Indiana Limestone Quarrymen's Association inaugurated five years ago an extensive research program at Purdue University covering the field of efflorescence and staining on Indiana limestone. This work was later taken over by the Architects' Service Bureau of the newly formed Indiana Limestone Co., when the Limestone Association was dissolved.

Efflorescence

Efflorescence is commonly found on brick masonry in various sections of the country. It is also found on concrete, sandstone, limestone, and frequently may even be found on dense materials like granite and marble. Numerous analyses of the water-soluble portion of samples of efflorescence have shown that most efflorescence consists principally of the alkali and alkaline earth sulfates, carbonates and chlorides. Sodium sulfate is found in varying quantities in efflorescent samples. In the alkali regions of the West particularly, efflorescence is mainly due to sodium sulfate. Sodium chloride occurs in some samples; but usually when this is found in large quantities its source can be traced to the mortar, where it has been added probably to prevent its freezing during the construction of the building.

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[1] Research Engineer, Indiana Limestone Co., Bedford, Ind.

Source of Efflorescence:

The sources of efflorescent salts embrace brick, cement, tile, sand, stone, mixing water, rain water, and the gases of the atmosphere. Brick and tile are usually low in salts that occur in efflorescent material. However, some contain relatively large quantities of sulfates of calcium, magnesium and perhaps sodium, and can be expected to be the source of much efflorescence. Most cements are known to contain soluble salts which may cause efflorescence. Gypsum, which is a constituent of most efflorescent samples, is added to cements to retard the rate of set. Sand, if clean, is in most cases unlikely to carry any efflorescent-forming material. Sodium and potassium salts would be most likely to be present in dirty sand or sea sand. Limestone contains some sulfate, but the amount present is hardly sufficient to cause more than superficial efflorescence. The acidic gases of the atmosphere, carbon dioxide, sulfur dioxide, and sulfur trioxide tend to cause efflorescence by acting chemically with the stone at its surface.

The ground at the base of a wall may be the source of efflorescent salts. Water tends to be absorbed by the stone from the ground, especially when no means are taken to break the capillary contact between the wall and the ground water.

A series of experiments on bricks from various sections of the country indicated that the mortars commonly used in setting brickwork generally cause more efflorescence than the brick itself. Portions of each series of brick were leached with carbonated water, and the leachings were analyzed. Bats from the same bricks were placed in an inch of water. Those bricks showing the largest amount of soluble material in the leaching experiment, produced, as would be expected, the most efflorescence when placed endwise in 1 in. of water. Efflorescence was found on only a few of the bricks when placed on end in water. When they were placed in water with a pat of cement on their lower end, however, they were all coated with a layer of white efflorescence. Some cements produced much more efflorescence than others. Several of the brick-setting cements were very bad in this respect.

Mechanism of Efflorescence:

Efflorescence is produced where evaporation of moisture takes place on the surface of any wall impregnated with a solution formed by dissolving salts from some part, or parts, of the wall masonry or adjoining construction. Crystallization of these salts from the solution takes place within the pores of the material adjacent to the face or on the surface of the wall, as efflorescence. This crystallization in the pore spaces of the stone will in many cases exert great pressure on their boundaries and cause surface spalling.

If a stone is saturated with moisture, and evaporation takes place more readily from one portion of the surface than from another, a flow of solution will take place towards that portion and the efflorescence will at that point be more pronounced. This accounts, in many cases, for the uneven distribution of efflorescence over the wall surface. The depth to which efflorescent salts are deposited in a stone surface is not very great and no cases have been observed in which it exceeds ½ in. If water washes the surface deposit off frequently, the salts cannot accumulate; but if they are not washed away there is an accumulation in and on the surface layer of the stone. Then crystal growth, changes in the size of molecular aggregates due to changing concentrations and metathesis, and frost action all combine to exfoliate the surface of the stone.

Crystal growth is based on the fact that larger crystals of a salt tend to grow at the expense of smaller ones when both are present in contact with a solution of the salt. The smaller crystals have a larger specific surface and solution pressure than the large ones and therefore dissolve more rapidly as the solution is diluted. This dilution would correspond to the stone being water soaked. When the stone begins to dry out again, recrystallization occurs with the larger crystals as nuclei. After passing through repeated cycles of recrystallization, great pressure is brought to bear on the boundaries of the pores and exfoliation often occurs.

When calcium carbonate changes to calcium sulfate, as happens when the oxides of sulfur act on the stone, an increase in volume occurs. This change of volume, along with the change of volume that occurs in the formation of the different hydrates of magnesium with changes in temperature, is sufficient to cause exfoliation.

The rate of the crystallization of salts is another factor to be considered in the disintegration of stone. It has often been noted that where efflorescence occurs on arches over doorways, etc., the soffits are disintegrated much more than the faces. This difference is undoubtedly due to the difference in the rate of evaporation of the moisture from the stone. On the face of the arch, evaporation is comparatively fast and the efflorescent salts are brought to the surface and deposited there as very small crystals which often only roughen the surface of the stone. On the other hand the soffits many times are badly flaked. Here on the lower side, sheltered from the direct rays of the sun, and also partially sheltered from drying winds, the crystals have time to grow before the moisture evaporates, and consequently exert pressure on the boundaries of the pores of the stone great enough to cause exfoliation.

This observation has been substantiated by an experiment performed on several pieces of stone. These stones went through fourteen cycles of immersions in a sodium-sulfate solution and dryings in the air at about 80° F. without apparently having much damage done to them. After the fifteenth immersion they were placed out of doors over night. The temperature was around the freezing point. In the morning the stones were all badly exfoliated. This was undoubtedly due to the fact that at this temperature the crystal formation was slower, and larger crystals were formed than would be formed at the higher temperature in the laboratory, thus causing the more pronounced disintegration.

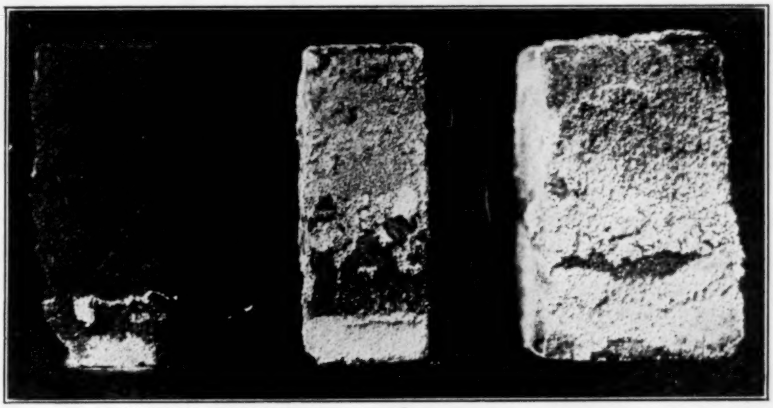
In all of these investigations, in no case have we been able to designate freezing as the cause of pitting or spalling. It is quite probable, though, that some disintegration may be due to ice forming in crevices started by the crystallization of salts. In northern climates, however, such as that found in Sweden, experiments and investigations seem to have shown freezing to do much damage to masonry.

Among the factors affecting efflorescence, the presence of water is of prime importance. Water is necessary in the mixing of mortars and analyses show that water might introduce in some cases an appreciable amount of efflorescent salts into the wall. If an excess of water is used in mixing the mortar for setting the stone and backing material, it will be drawn into the stone or backing material and will carry with it the original soluble material it contained along with any soluble material it may have dissolved from the mortar. When seepage occurs through the wall this material will at once be picked up and deposited at the surface of the stone as an efflorescence. The most common sources of water seepage which are responsible for efflorescence include leaky gutters and parapet walls, leaky drain connections, open vertical joints in copings and cornices, poorly drained balconies or porch roofs and seepage or absorption of moisture from the ground which will tend to rise in the wall by capillarity.

Prevention of Efflorescence:

The time to start guarding against efflorescence, in a properly designed building, is during its construction. If, during the construction of the building, the walls are exposed to rain or water from other sources the whole structure will become water soaked. In time the excess water escapes at the exposed surface of the wall by evaporation and the usual phenomena of efflorescence occurs, whereas, if the wall masonry is kept free from moisture seepage both during construction and after construction, no efflorescence will occur.

Considerable work has been done in trying to prevent efflorescence by the use of waterproofing compounds. No waterproofing treatment that seems suitable for this purpose has been found.



No. 1 No. 2 No. 3

FIG. 1.—Shows the Action of a 5-per-cent Sodium Sulfate Solution on Waterproofed Stone After Being in Test One Month.

Each of the stones in Fig. 1 was waterproofed on all six sides with two coats of well-known waterproofing compounds and after drying were placed in about 1 in. of a 5-per-cent sodium-sulfate solution. The waterproofing compounds on Nos. 2 and 3 broke down within 2 days. The waterproofing compound on No. 1 did not break down completely after being in test one month. However, there is some disintegration on the stone that is not visible in the photograph.

In a series of tests conducted on colorless and other waterproofing materials, embracing twenty-two different products, it has been found that a waterproofing preparation that may be effective against moisture alone is not always effective against an alkaline seepage. And again, a waterproofing preparation that is more or less effective against an alkaline seepage is not always the one that is most effective against a solution of sodium sulfate or possibly other salt solutions. These preliminary conclusions were arrived at from the results obtained in the following experiments: Three series of assay crucibles were prepared by painting the outer surface of a crucible in each series with two coats of one of the waterproofing compounds. The two coats were put on at twenty-four hour intervals and the last coat was allowed to dry before being put into test. Then 50 cc. of a 5-per-cent solution of sodium carbonate was added to the crucibles in the first series, 50 cc. of a 5-per-cent solution of sodium sulfate was added to the crucibles in the second series, and the crucibles of the third series were filled with a saturated calcium-hydroxide solution. More solution was added to the crucibles when they became dry.

While these experiments were not as complete as they might have been, and may not seem to approach actual conditions found in a building, they nevertheless indicate that no one type of waterproofing is best for use against all types of moisture seepage. Several of these compounds which were known to stand up against water alone broke down when subjected to the action of the salt solutions.

A series of stone specimens, waterproofed on all six sides with a comprehensive selection of well-known waterproofers, when placed in a 5-per-cent solution of magnesium and sodium sulfates, were disintegrated within a short time. The surface of these stones did not crumble but came off in large flakes. This same phenomenon was noted in a large building, the stone of which had been waterproofed on its exterior face and backed with material known to be very high in soluble efflorescent material. In this case, as in the above experiment, crystal growth occurred just back of the dense skin formed by the waterproofing compound and forced off, by wedging action, large flakes of stone. Flaking has also occurred in instances where the stone has formed a hard outer skin by natural weathering conditions. This type of exfoliation is quite frequently found in England on the soft Headington and Doulting stones.

Two important factors in preventing the occurrence of efflorescence are as follows: First, the use only of materials that contain very little soluble material; and second, the following of good construction practices so as to prevent the possibility of any moisture seepage into the wall. The first is hard to attain because of the universal presence of soluble salts in building materials; however, such materials that are known to contain a large amount of soluble matter should not be employed whenever their use can be avoided. Another method that has been found very effective, for the time being at least, is to parge the back of the stone with a mortar made from an integrally waterproofed cement. The success and general commercial use of this method of attack depends, of course, on at least two factors: First, the permanence of the waterproofing material incorporated in the cement; and second, the price at which such a waterproofed mortar can be prepared.

STAINING OF INDIANA LIMESTONE

The staining of Indiana limestone involves a mechanism similar to that involved in the production of efflorescence, but has certain distinctive characteristics. The usual brown stain which occurs on walls during construction or just after construction may be called an alkali stain, because it is produced by fixed alkalies dissolving organic salts which are found in Buff Indiana limestone and depositing them on the surface of the stone as the solution carrying them evaporates.

Salts of sodium or potassium or both occur in greater or lesser amounts in all portland cements and may also occur in other wall materials. Whenever any material containing these salts is in capillary contact with the wall, the salts will be dissolved where water is present and later, as the water evaporates, be deposited as an efflorescence at the surface of the wall. If conditions are favorable, an undesirable brown stain is also deposited along with the efflorescence on the surface of the Buff Indiana limestone, a grade which is characterized by the presence of traces of organic matter that has been oxidized by the oxygen in the atmosphere to form an acid-like substance similar to humic acid in soil. The alkaline solution in passing through the stone reacts with the organic material and forms a soluble organic salt which is deposited as a brown discoloration on the surface of the stone where the water evaporates. This discoloration is not permanent and where exposed to the action of the weather will disappear in a short time, leaving little or no trace behind. On the soffits of arches, however, or on other places not exposed to the action of the weather, it will frequently remain several years.

Cause of Staining:

It has been definitely determined by several experiments that mortars frequently used in setting stone and the backing material are the primary causes of the brown stain found on Indiana limestone.

Before anything of a definite nature was known of the brown stain found on some Buff Indiana limestone, it was observed that wherever a Buff piece of the limestone was in contact with the ordinary portland cements the brown stain was formed. In the laboratory it was noted that some of the cements produced a heavy brown stain on Buff stone while others produced practically no stain. This test was made by placing a Buff piece of stone on a neat pat of cement which was kept moist with distilled water. The moisture extracted the soluble alkalies from the cement and was then drawn up into the more porous stone where the alkaline solution dissolved the organic material and deposited it on the surface as a brown discoloration when the solution evaporated. A comparative list of the cements was made from these results with the high-staining cements at the top, ranging to the low-staining cements at the bottom. Samples of over twenty different cements and about a dozen brands of lime were obtained from different parts of the country for this and the other tests that are discussed below.

It was found that the brown stain could be produced by standing a piece of Buff stone in a solution of the carbonates or hydroxides of sodium or potassium. The characteristic brown stain could never be produced by any other means, although the stain seemed to be enhanced when any soluble sodium or potassium salt was added to a non-staining cement and pasted on the lower end of a piece of Buff stone placed on a moistened blotter. This was probably due to base exchange between the sodium sulfate and the calcium carbonate, thus forming sodium carbonate which, under proper conditions, is an active agent in producing the brown stain.

Later, the cements mentioned above were analyzed; they were set up into pats, the pats were pulverized and leached with water. The leaching water was analyzed and from its alkali content a tentative relative staining list of the cements was made. This list checked very closely the results obtained in the first series of experiments in the laboratory.

Two steam-tight buildings with vertical panels of stone, separated by strips of wood, forming the face, were constructed in order to determine what effects different mortars would have on stone in actual construction conditions. Twelve samples of stone, some Gray and some Buff, selected from different parts of the Indiana limestone district, were used in these faces. In the first building, thirty different cements or combinations were used to set and parge thirty vertical panels, each of which contained one of each of the twelve stones mentioned above. One panel in this building was set with rubber between each stone and was not parged. This served as a blank. Each of the other panels was set and parged with mortar made from Ottawa sand and a particular brand of cement.

The second building was constructed the same as the first, but the panels were set in various brands of lime, a magnesite cement, a special plaster and some other combinations. Several of these panels were backed with brick set in a cement that was known to produce heavy staining.

The structures were allowed to stand for some time and no appreciable amount of staining or efflorescence occurred on the face of the stones. Steam was then turned on in these buildings and allowed to remain on until the stones were saturated with moisture, then it was turned off and the stones were allowed to dry. Efflorescence and stain appeared on the Buff stones in nearly all of the panels. Some of the panels were badly stained, some had an intermediate amount of stain and others had practically no stain at all. Quite a few of the panels contained much efflorescent material on the face of the stones.

Some of the cements produced such a small amount of staining that they could be classified as non-staining. Among these classified as practically non-staining were a puzzolan, two or three brands of white portland cements and one waterproofed white portland cement. The ordinary gray portland cements produced appreciable amounts of staining and some of them produced very much efflorescence. One brick-setting cement gave results that would almost classify it as non-staining, while two others produced very much staining and considerable efflorescence.

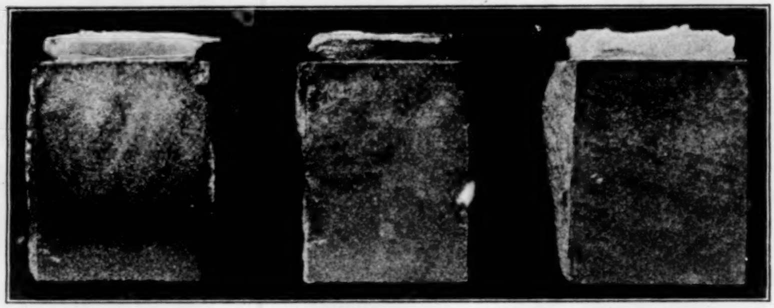
The stones in the blank panel were not changed. Only a few of the panels set in the lime mortars showed a little efflorescence. None of them produced an appreciable amount of stain.

Another list showing the relative staining power of the cements was prepared from the results obtained from the panels and it closely checked those obtained in the laboratory.

From these results the various cements were classified as to their relative potential staining ability and by analyzing the leachings from pats and by making a simple laboratory test, a sack of any of the portland cements can now readily be classified according to its staining ability. In this manner the bad staining cements can be segregated and if some means is not found to lower the staining power of these cements, they will have to be classed as being undesirable for use on work in which seepage from the cement can possibly come in contact with the stone. Some portland-cement manufacturers have already done some experimentation in an effort to produce a truly non-staining cement and in one case, at least, it seems, will likely succeed. The non-staining cements that are really non-staining will undoubtedly have preference over the other cements for use in conjunction with stone.

Method for Predicting Amount of Stain Produced by a Cement:

A simple laboratory test has been developed by which nearly anyone can determine approximately the staining power of a cement. The first thing to do is to get a good supply of Buff stone of one color and texture, from the same block, if possible. The stone is broken into a convenient size and a pat of neat cement (about 100 g. of cement mixed with approximately 35 cc. of water) is placed on the upper side of the stone and a depression is made in it and kept filled with water. The staining power of the cement is determined by the amount of brown stain that is formed on the stone. Two or three tests should be run in series as checks so that any variation that may occur in one piece of stone can be detected and results thereby obtained corrected. The stain obtained by the cement being tested should be checked against those obtained from a non-staining cement and a cement that is known to produce very much stain. If the cement is going to produce a stain, it will appear within a week.



No. 1 No. 2 No. 3

FIG. 2.—Showing Method of Testing Amount of Stain Produced by Cement of Unknown Staining Quality.

A 3-in. piece of stone, broken from a long 4 by 4-in. piece, is of a convenient size to handle and is ample in size to show clearly the character of stain likely to be produced by the cements. Figure 2 shows the working of this test very clearly. Stones 1, 2 and 3 were broken from the same block; No. 1 was capped with a known bad-staining cement, No. 2 was capped with a cement of unknown staining quality, while No. 3 was capped with a known non-staining cement. Only a very slight stain was produced on No. 2 and the unknown cement would be classified as practically non-staining.

Prevention of Stain:

Several methods have been tried for preventing the occurrence of stain but where improper mortar has been used, and where improper construction practices have been followed allowing the wall to become watersoaked, no method for its prevention has been found which has proved to be entirely successful. A comprehensive list of waterproofings of the various types on the market were experimented with and some of them were effective in preventing the occurrence of staining for some time, but none were permanent.

Experiments were performed in the laboratory with each of these waterproofing compounds subjecting each to two series of tests. In the first test the waterproofing compounds were applied in a manner that was intended to prevent the penetration of moisture seepage into the stone. This corresponds to waterproofing the back of stonework. Two coats of each waterproofing compound were placed on the upper horizontal surface of the stone at 24-hour intervals. Pats of staining cement about one inch thick with a depression in their centers were then placed over the thus waterproofed surface. The depression in the cement was kept filled with water. All of the commercial waterproofings tested broke down and allowed the alkaline moisture from the cement to seep through and stain the stone. The cement pats were not allowed to come within an inch of the edge of the stone, so as to prevent the possibility of alkaline seepage from running around the edges of the waterproofed portion.

In the second test the waterproofing compounds were applied in an entirely different way, intended to prevent moisture once in the stone from seeping out towards the treated surface and evaporating on the face. This corresponds to waterproofing the face of the wall. In this test one vertical surface in a series of the stone specimens was waterproofed in the same manner as the horizontal surfaces were in the first test. The pat of cement was placed on the upper horizontal surface as in the first test, and kept filled with water. The same waterproofing compounds, when tested in this manner, did not prevent the occurrence of the stain on the treated surface.

These experiments and actual field observation lead us to the conclusion that the prevention of this type of stain is a matter of choosing suitable non-staining cements for mortar and of keeping the wall dry, both during and after construction.

Removal of Stain:

As stated before, the brown stain will usually disappear in a few months' time, if the wall is dry and exposed to the action of rain and sunlight, leaving practically no trace behind. In some cases it is desirable to remove the stain artificially. This can be done in two ways. The first method is to blow the stain off with a blast of steam under 125 to 150-lb. pressure and to then wash the treated surface immediately with a 5-per-cent formic acid solution. The second method consists of treating the surface with a poultice moistened with a 5-per-cent solution of sodium carbonate. The poultice treatment is most useful where the stains are restricted to a small area and the steam treatment where large surfaces are to be cleaned.

In employing the first method for removing the brown stain, it is only necessary to play the jet of steam over the stained portion until the stain is removed. In very resistant cases some of the stain is not removed in this manner, and it is necessary to wash the remaining stain with a 5-per-cent solution of sodium carbonate and allow it to remain for about 15 minutes before resteaming. The wash with the formic acid solution prevents the recurrence of the stain, if the interior of the wall is dry.

In the second method, paper pulp is prepared by digesting and blowing newspaper into a pulp with steam. It is then mixed with enough fire clay to make a somewhat plastic mass that is too dry to spread with a trowel. This mixture is then brought to the proper plasticity for spreading with a 10-per-cent solution of sodium carbonate. This alkaline poultice placed over the stain dissolves the stain and this solution is drawn into the poultice as the moisture evaporates at its exposed surface. The poultice is removed within 24 hours, leaving a clean surface. This poultice is followed immediately with one made like the alkaline poultice with the exception that it is given its final plasticity by moistening with a 10-per-cent solution of formic acid. This acid poultice prevents the recurrence of the stain the same as the acid wash does in the first method.

When the character of the brown staining was first determined, it was observed that the stain could be removed by an alkaline scrubbing but within a very short time more organic material would be brought to the surface and the stone would be as badly stained as before. Various acids and other chemicals were tried as an after wash to prevent the recurrence of the stain but none of the washes, with the exception of an acetic acid wash and a formic acid wash, did any good. It is not known just why these acids should prove better for this use than other acids. Acetic acid discolors the stone somewhat, so formic is used in the treatments for the removal of stain.

Any treatment for the removal of stain which removes the fine powdery stone dust formed in the surfacing of the stone will leave a spot that stands out in contrast with the rest of the building and this contrast can only be obliterated by a year or so of weathering. For this reason many people prefer to allow the stain to disappear on its own accord, which generally occurs in a reasonably short time.

DISCUSSION

Mr. Palmer.

MR. L. A. PALMER[1] (*presented in written form*).—The paper by Mr. Lee Huber is one of considerable interest. I am not familiar with matters pertaining to the cause or prevention of brown staining which he has discussed but I have had some experience with white efflorescence. I wish to make the following comments concerning the latter.

With reference to the source of efflorescence, the acidic gases of the atmosphere, sulfur dioxide and sulfur trioxide, also carbon dioxide, dissolve in rain water, snow water, etc., and if such water, thus rendered acidic, enters a crack in a wall produced by settlement, the rate of disintegration of the mortar, stone, etc., is accelerated. Vertical joints in copings, cornices, etc., are particularly vulnerable to this attack. Prompt pointing of all such vertical joints and cracks due to settlement is very necessary.

In an investigation sponsored by the American Face Brick Association at the U. S. Bureau of Standards, 288 panel tests, extending over a period of six months, were made. Thirty-two types of face brick and four types each of lime, bricklayers' cement and portland-cement mortars were used in the construction of these panels. Eighteen of the 32 types of face brick did not develop any efflorescence during a six months' continuation of severe tests made on the brick alone and out of contact with any mortar. These same eighteen types of brick, however, did show efflorescence in the panels containing he various mortars.

The lime and bricklayers'-cement mortar contributed more to the efflorescence than did the portland-cement mortars. Lime itself is sparingly soluble.

Any soluble material in well-burned brick is usually calcium sulfate. Burning decomposes other salts practically completely. The amount of undecomposed calcium sulfate in a well-burned brick is usually negligible. A brick having no more than 0.05 per cent of soluble sulfuric anhydride is safe from the standpoint of efflorescence.

Portland cement producers have claimed that the gypsum added to the clinker to retard the rate of setting is rendered insoluble by

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[1] Research Associate, American Face Brick Association, U. S. Bureau of Standards, Washington, D. C.

Mr. Palmer. conversion to calcium-aluminum sulfate. The writer is ignorant of any existing data proving that this reaction proceeds to completion. It is conceivable that it may be reversible and that some of the gypsum may remain as such after the cement is set.

In respect to the prevention of efflorescence, the writer is in very thorough agreement with the author. Of 250 very noticeable instances of efflorescence studied by two independent and more or less trained observers, it was concluded (independently) that with two definite exceptions, every case indicated that efflorescence could have been prevented through proper design, construction and maintenance of the buildings showing efflorescence.

Concerning the method of predicting the amount of stain produced by a cement, I wish to suggest that any manufacturer's cement is subject to a considerable degree of variability. A manufacturer's cement produced to-day may not produce a stain whereas that produced by the same cement manufacturer to-morrow may produce a noticeable stain.

Mr. Dutton. MR. H. H. DUTTON[1] (*presented in written form*).—Mr. Huber has presented various features of the appearance of efflorescence and brown stains on Indiana limestone which are well substantiated by field observations made by the writer. Comment will be made on some of the protective methods which are not included in his paper.

It may well be considered an axiom in the use of masonry that a "dry wall never stains." By "dry wall" is meant a wall into which no additional moisture has penetrated other than that included in the mortar used for setting the stone and backing material. The amount of water in the setting mixture is rarely sufficient to cause trouble, but if outside sources provide more moisture, staining and efflorescence may result.

In construction practice efforts are frequently made by conscientious contractors to protect the masonry. Various schemes have been employed and of these probably the least expensive and most effective is that embodying the use of 50-ft. strips of waterproof canvas about 4 or 5 ft. wide, which are laid over the top of the unfinished wall each night and when bad weather threatens. These canvas strips can be made up by awning manufacturers or sail makers. The labor cost of such a method is not excessive and the canvas strips are usually long lived if carefully handled. One contractor is known who has employed this method with success for the past

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[1] Research Associate, Indiana Limestone Company, U. S. Bureau of Standards, Washington, D. C.

Mr. Dutton. $2\frac{1}{2}$ years. He has stated that the expense involved in this method of protection has been well worth while in consideration of the satisfaction of the architects and owners when the jobs were so cared for.

In the case of concrete construction where the floors are poured ahead of the stone veneered enclosure walls, rain may cause an accumulation of water on the floor slabs and this water may run off over the masonry walls. This wash from portland-cement concrete may often cause stains and efflorescence and should be prevented where possible by providing some means of draining the water off into the building or at least away from the stone work.

One method which has been noted as successful for this situation is the placing of small dikes of sand along the edge of the floor two or three feet back from the wall line so as to prevent the passage of water from the edge of the floor slab onto the masonry. The cost of this procedure is small and the results are good in most cases.

These two schemes of protection are suggested as having been successful in actual construction work and economical in labor and material costs.

Concerning waterproofings and their effect, it has been noted that where it is possible to waterproof the interior face of the stonework before the backing is laid up against it, good results are often obtained with some of the coal-tar and asphaltic waterproofings. In this situation the waterproofing would be applied over the stone surfaces and the joints as well, so there would be no places where water might get through unless actually penetrating the waterproofing.

Along this same line it may be noted that when stone work is to be placed against concrete walls, the concrete should be thoroughly coated with a coal-tar or asphaltic waterproofing as a protective measure.

It may be of interest to relate how one contractor has followed a scheme of waterproofing the masonry after it has been set up and using the stone veneering and brick backing at column locations to make three sides of the form for the structural columns. In two buildings recently erected, the stone and brick enclosure walls were carried up one story ahead of the concrete frame and after waterproofing the stone and brick work with two coats of coal-tar solution, portland cement concrete was poured against the stone and brick which made up three sides of the form for the columns as already mentioned above. This practice is not one which should be generally recommended, but it may be mentioned because of the fact that the waterproofing over the surfaces and the joints of the stone and brick work was effective in preventing any staining elements coming through

Mr. Dutton. to the stone even though the wet concrete was poured immediately against it.

Mr. Kessler. MR. D. W. KESSLER.[1]—As I understood the paper, the statement was made that Gray stone is not affected by brown stains. If my memory does not fail me, our tests would not quite agree with that. In some cases we did get considerable stain on Gray Indiana limestone by using alkaline solutions, and such a stain was more permanent than stains on the Buff stone. I would like to ask if that is borne out by observations in practice.

Mr. Huber. MR. LEE HUBER.[2]—The stain discussed in this paper refers to brown stain on the Buff stone. It is well known that a cement is liable to produce a so-called gray stain on a Gray stone where it is not well seasoned, but in all our experimental work, in all our observations, I have never seen brown stain on a Gray stone.

Mr. Carney. MR. L. J. CARNEY[3] (*by letter*).—Referring to the paper by Mr. Huber, we wish to call attention to causes and conditions other than the backing-up brick mortar that should not be lost sight of as causes of staining limestone. Mr. Huber has touched lightly on a number of these other important causes but places more stress on the wall backing materials, and particularly the cement in the mortar, as the chief cause for efflorescence and staining. There is no doubt that certain brands of cement will more easily stain Bedford stone than others, but conceding that the results of the tests made would be corroborated in actual building practice, there are too many other elements entering into the staining of Bedford stone to confine the blame almost entirely to the cement in the backing-up wall.

In the paper, Mr. Huber says, "sand, if clean, is in most cases unlikely to carry any efflorescent-forming material." Unfortunately, a very large amount of dirty sand, much of it dug out of the ground at the building site, is used to-day. This is particularly true in the large cities where it would be expected that Bedford stone is most widely used. Sand forms by far the largest part of the brick mortar and since it is possible for it to contain impurities, it should be given more consideration. If exhaustive tests are carried on to determine the relative staining quality of a number of brands of cement, it is even more important to determine what sand to use since more sand than cement is used in the mortar.

It is also quite important that the water used in the mortar be free from anything that might cause efflorescence or staining. Inasmuch as all brick, and particularly common brick, used in backing up

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[1] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

[2] Research Engineer, Indiana Limestone Co., Bedford, Ind.

[3] Treasurer, The Carney Co., Mankato, Minn.

Mr. Carney. walls are not free from efflorescence forming material, they too should be tested so as to eliminate the undesirable makes when using Bedford stone.

Mr. Huber states that the ground at the base of the wall may be the source of efflorescent salts. The trouble is not always confined to the wall of the building. In many cases blocks of Bedford stone have been allowed to lie on the ground unprotected before being placed in the wall, and have become badly stained in that way.

Mr. Huber also states that the "acidic gases of the atmosphere, carbon dioxide, sulfur dioxide, and sulfur trioxide tend to cause efflorescence by acting chemically with the stone at its surface." This being true, the staining of Bedford limestone would seem to be beyond control, and not dependent solely on the character of the cement.

Bedford limestone is quarried in a number of places over quite a wide district. There is also a vast difference in the various ledges of stone in a single quarry. It is, therefore, quite possible that stone from one quarry, or from certain parts of a particular quarry may stain more easily than others.

The problem in the matter of staining of Bedford limestone no doubt prompted the tests referred to in Mr. Huber's paper. The earnest efforts to attempt to eliminate the cause of staining can be appreciated. The blame, however, should not be placed almost entirely on the cement in the mortar of the backing-up brick walls. Until the stone itself can be guaranteed as uniform and free from staining, the sand and water guaranteed clean and there can be no possible chance of stain originating from the brick, the ground or the atmosphere, statements attributing staining to the use of cement in mortar are not justified.

Mr. Huber, in referring to the tests on lime mortar, states, "Only a few of the panels set in the lime mortars showed a little efflorescence. None of them produced an appreciable amount of stain." It must be considered that a straight lime mortar is seldom used in practice, and particularly in large buildings, the class of work in which limestone is more widely used. It would not be wise to consider impairing the strength of the wall by eliminating the one binding material—cement.

Mr. Huber. MR. HUBER (*author's closure by letter*).—Replying to Mr. Carney's discussion, all of the points brought up in this discussion in regard to staining and efflorescence on stone were discussed briefly in the paper. In a paper of this size all of the minor points directly or indirectly related to staining and efflorescence cannot be fully discussed, and it must not be construed from the fact that they have been only very briefly touched upon that they have been overlooked.

Mr. Huber. In regard to sand, Mr. Carney states that much dirty sand is used to-day and since sand forms the greatest part of the mortar, it should be given more consideration. We have run a few tests on different sands and those that we have run on sand that was suspected of producing stain proved that it contained no material which in any way could cause staining. The use of dirty sand is not a new practice. In fact, dirty sand was probably used in building mortars before the advent of portland cements, yet we never experienced at that time any of the brown stain trouble now known as cement stain. It has also been definitely proved that with two mortars, one made from a staining cement and the other made from a non-staining cement, the same kind of sand being used in both, the one made from the staining cement will stain the stone, while the other mortar will not. Further tests with various sands indicate that the sand as compared with the cement is a much less important, almost negligible factor in production of any serious staining. It is, however, admitted as mentioned in the paper that sand may contain soluble material which causes efflorescence.

Mr. Carney states that brick too should be tested so as to eliminate the undesirable makes or kinds when using Bedford stone. By undesirable makes, I assume that Mr. Carney means those high enough in soluble material to produce efflorescence. In that, I heartily agree with Mr. Carney that the brick should be tested and if found to contain enough soluble material to produce much efflorescence, precautions should be taken not to use them where they are subjected to moisture seepage. This point is one, however, that is just as serious where brick alone are to be used without limestone. I believe the brick manufacturers have done and are doing much successful research work on this very problem. We ran a rather extensive experiment with burned brick collected from various sections of the country and found that while some of them contained enough soluble material to produce efflorescence, none of them contained enough free alkali to produce stain, even when subjected to the most favorable condition for producing stain.

It is also stated that the water used in the mortar should be free from anything that might cause efflorescence and staining. We have time and time again run tests on the same kind of buff limestone, trying to produce stain both by tap water and by distilled water and have never been able to do so. From the analysis of water from various sections of the country, except possibly from rather strongly alkaline streams, I can see no reason why water should stain the stone. Again, some waters which do not cause

Mr. Huber.

stain will be found to contain enough soluble material to produce efflorescence on any masonry structure in which they may be used where the conditions for its production are present.

In the fourth and fifth paragraphs of Mr. Carney's discussion, he should distinguish more clearly between efflorescence and staining. In the paper it is stated that the ground at the base of the wall may be the source of efflorescent salts and that the acidic gases of the atmosphere, carbon dioxide, sulfur dioxide and sulfur trioxide tend to cause efflorescence by acting chemically with the stone at its surface. In both cases Mr. Carney connects these with the staining phenomena, which is not the case. Efflorescence is produced by any soluble salts being deposited on the exposed surface of the stone or other masonry, whereas the stain described in this paper is caused only by the carbonates and hydroxides of the alkali metals. From this it will be seen that the source of this staining material is limited to a very definite few structural products.

Mr. Carney states that, "There is also a vast difference in the various ledges of stone in a single quarry. It is, therefore, quite possible that stone from one quarry, or from certain parts of a particular quarry may stain more easily than others." This may be quite true but has nothing to do with the potential staining agent (free alkali) that may be present in the wall to bring out as a stain what alkali soluble organic matter there is in the stone.

In the next to the last paragraph, Mr. Carney states: "The blame, however, should not be placed almost entirely on the cement in the mortar of the backing up brick walls. Until the stone itself can be guaranteed as uniform and free from staining, the sand and water guaranteed clean and there can be no possible chance of stain originating from the brick, the ground or the atmosphere, statements attributing staining to the use of cement in mortar are not justified." It was not our desire to place the cause of stain on any particular material. However, all of our field observations and all of our laboratory results show that the brown cement stain on Buff Indiana limestone is invariably caused by the relatively high free alkali content of some certain cements. Cement is a practical necessity of modern construction; but in this we wish to encourage the use, in conjunction with Indiana limestone, of non-staining cements, that can be satisfactorily and economically used.

THE CALIBRATION OF EXTENSOMETERS

By R. L. Templin[1]

Synopsis

This paper discusses in some detail a new design of calibration device suitable for calibrating extensometers arranged for measuring deformations on gage lengths varying from ½ in. to 10 in., inclusive.

This calibration device indicates total deformations to the nearest 0.000002 in. The scheme used to obtain such refinement of measurement has as its basis the use of a differential-thread micrometer screw and a 5-to-1 lever supported on a system of plate fulcra. The special micrometer screw can be calibrated by means of a suitable jig and Johansson blocks, or a precise end-measuring machine or suitable interferometer.

Errors due to temperatures, design, workmanship, and manipulation are discussed.

Calibration data pertaining to the micrometer screw are included together with results obtained on a few different types of commercial extensometers.

There are available to-day many kinds and varieties of extensometers intended for use in measuring the comparatively small deformations occurring in materials, especially metals, as a result of applied loads or forces. These instruments are usually arranged to measure deformations or strains occurring in definite original gage lengths of 1, 2, 3, 4, 6, 8, 10 or more inches, but of these gage lengths the more commonly used in this country are the 2 in. and 8 in.

With very few exceptions all of the commercial extensometers, especially those used for routine tests of metals, depend upon a mechanical system of levers, gears, or both as a part or all of their mechanism for magnifying the minute deformations so that they can be readily measured. In some types of extensometers the lever system is supplemented by electrical or optical systems to further increase the magnification, sensitivity and presumably the accuracy of the instruments.

Experience in the use of a wide variety of different makes, types and sizes of extensometers and strain gages has definitely shown that these instruments should be calibrated when received from the manufacturer and at comparatively frequent intervals during their subsequent routine use, if the data obtained with them are to be acceptable. This is particularly true if the instruments are used in determining proportional limits and moduli of elasticity of materials. Certainly an extensometer should be calibrated after an adjustment of its multiplying system has been made or after the instrument has been damaged and repaired.

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

A rather simple apparatus for calibrating extensometers was described by the author some years ago.[1] The calibrating device depended primarily upon a calibrated micrometer screw capable of being read to the nearest 0.0001 in., plus the assumption that the mechanical lever system of an extensometer would constantly magnify any errors in the extensometer to a value where they could be readily measured with the calibrated micrometer screw. Although this device has proved satisfactory within certain limits for checking extensometers, yet the trend of progress in testing materials has continued to demand greater sensitivity and closer tolerances in measuring deformations, with the result that the errors in extensometers must be more definitely known and evaluated. The problem is thus presented of providing a calibrating instrument which will have a suitable sensitivity, accuracy, range and manipulatability that will permit of readily checking or calibrating a wide range of sizes and types of extensometers. Obviously, any such instrument must be capable of being checked by some measure of a still higher degree of accuracy that will be acceptable as a primary standard.

The final design of instrument selected involved a differential micrometer screw used in conjunction with a 5-to-1 lever. A sketch of the instrument is shown in Fig. 1 and a photograph of it in Fig. 2. The two threads on the micrometer screw are 40 and 50 per inch, respectively, and the disk between them is graduated into 500 parts or divisions. The nuts into which the threads screw are split into six segments each and can be adjusted by means of other nuts with tapered threads so that there is very little lost motion, if any, in the micrometer screw threads. These nuts in turn are secured, one to the base of the instrument and the other to one end of the lever, just referred to, by means of hardened and ground pointed screws. The actual lengths of the lever arms were made of sufficient size that the usual errors obtained in good machining work would produce negligible errors in the readings of the instrument. That is, the lever arms were made 15 in. and 3 in. long, respectively, so that an assumed error of ±0.001 in. in the shorter would give an error of ±0.03 per cent;

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[1] R. L. Templin, "An Extensometer Calibrating Device," *Chemical and Metallurgical Engineering*, August 10, 1921, p. 248.

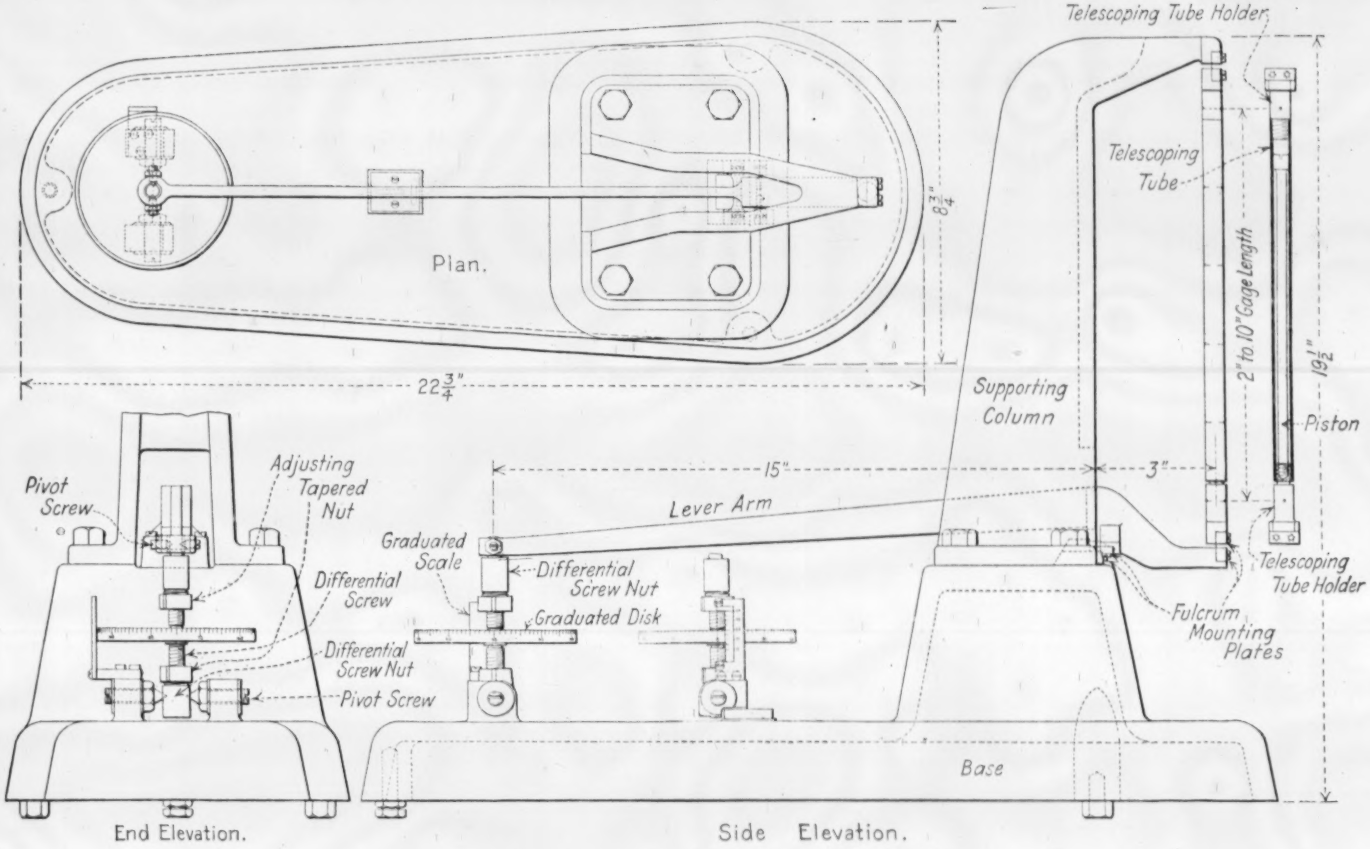
Plan.
$8\frac{3}{4}$"
$22\frac{3}{4}$"
Telescoping Tube Holder
Telescoping Tube
2" to 10" Gage Length
$19\frac{1}{2}$"
Piston
Supporting Column
15"
3"
Lever Arm
Telescoping Tube Holder
Fulcrum Mounting Plates
Graduated Scale
Differential Screw Nut
Graduated Disk
Base
Pivot Screw
Adjusting Tapered Nut
Differential Screw
Differential Screw Nut
Pivot Screw
End Elevation.
Side Elevation.

FIG. 1.—Extensometer Calibrating Device Assembly.

and much less in the longer lever. The lever was mounted on plate fulcra 90 deg. apart so as to permit of rotation only in one plane.

The telescoping tube and piston arrangement to which the extensometers are attached is supported at either end by other plate fulcra which are in turn fastened to the short end of the 5-to-1 lever and the supporting column of the instrument. The double-ended piston is lapped into the tube so as to have no lateral lost motion and yet function easily longitudinally.

The differential micrometer screw has been checked by the U. S. Bureau of Standards using selected Johansson blocks having known constants. In carrying out this work a special jig was used to hold the screw and its nuts in the same relative position that they occupy in the complete instrument. The relative movement of the nuts was determined to the nearest 0.00001 in. and the readings were found to repeat within ±0.00003 in. The results obtained indicated an average error of approximately −0.3 per cent throughout the first half of the travel of the screw, which is the range used in nearly all cases. (See

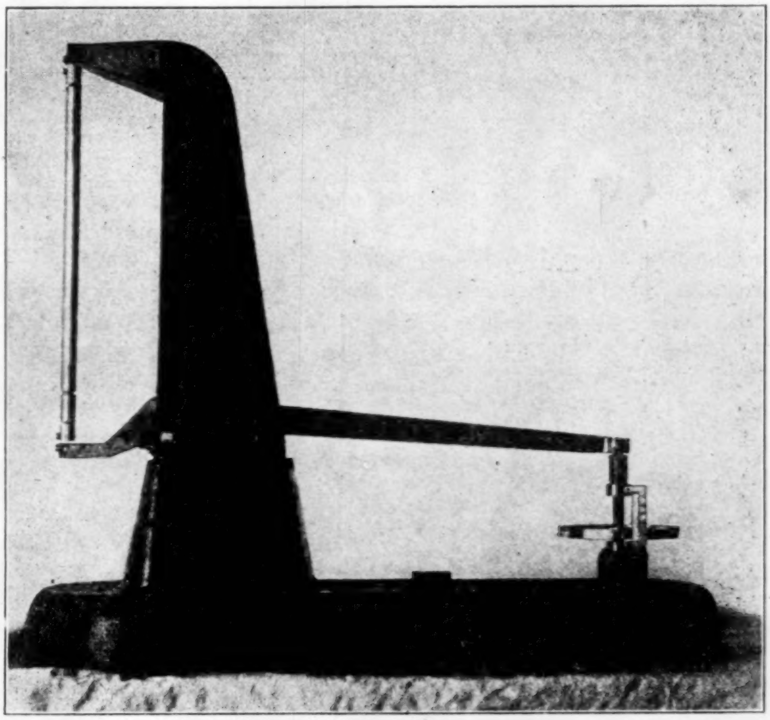

FIG. 2.—Extensometer Calibrating Device.

Table I and Fig. 3.) This error added to that previously indicated for the lever of ±0.03 per cent would give a possible total error of −0.33 per cent. Only the major error of −0.3 per cent is used as a correction for the instrument readings.

TABLE I.—CALIBRATION OF DIFFERENTIAL MICROMETER SCREW.

Reading of Micrometer	Actual Displacement of Nuts, in.	Reading of Micrometer	Actual Displacement of Nuts, in.
0.000	0.000 00	0.024	0.023 92
0.005	0.004 97	0.025	0.024 92
0.010	0.009 96	0.026	0.025 93
0.015	0.014 96	0.027	0.026 91
0.016	0.015 97	0.028	0.027 89
0.017	0.016 96	0.029	0.028 89
0.018	0.017 95	0.030	0.029 88
0.019	0.018 95	0.035	0.034 89
0.020	0.019 95	0.040	0.039 88
0.021	0.020 95	0.045	0.044 87
0.022	0.021 94	0.050	0.049 86
0.023	0.022 93	0.100	0.099 65

NOTE.—The screw will repeat its reading to about 0.00003 in. The accuracy of the values given in the table is well within that amount.

The total range of the micrometer screw is 0.2 in., which gives a total range for the instrument of 0.04 in. One division of the index corresponds to one revolution of the graduated disk on the micrometer screw or 0.001-in. movement of the piston in the tube to which the extensometers are attached. One graduation on the disk corresponds to $\frac{1}{500}$ of this value or 0.000002 in., and half divisions can be readily estimated. The readings obtained with the instrument check the total deformations as measured by an extensometer.

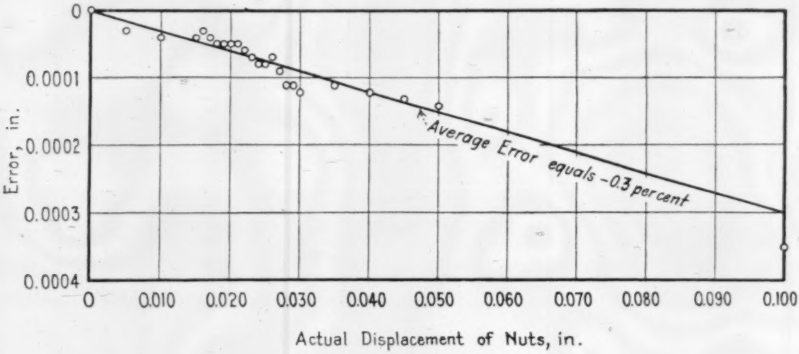

FIG. 3.—Graph of Calibration of Differential Micrometer Screw.

A consideration of the details of the instrument will show that temperature effects on the accuracy of the readings obtained with it are very small indeed, provided no appreciable changes in temperature occur during the actual calibration of an extensometer. Since the instrument is always used in a room free from draughts and since the checking of an extensometer requires only from five to thirty minutes, the temperature errors have been considered negligible. The instrument, therefore, is at least ten times more sensitive than the better commercial extensometers and with the accuracy indicated, may be used with considerable assurance in calibrating such extensometers. In Table II are given data pertaining to some commercial extensometers which will serve to point out the values that must be checked when calibrating such instruments.

TABLE II.—SMALLEST DEFORMATIONS MEASURED BY VARIOUS EXTENSOMETERS.

Extensometer	Value of Total Deformation			Total Range	
	Inch per Division	Smallest Estimation			
		Division	Inch	Divisions	Inch
Ewing (Combined Microscope and Lever Extensometer)	0.0002	0.2	0.000 04	140	0.028
Amsler-Martens Mirror Extensometer	0.0001	0.2	0.000 02	240	0.024
Riehlé Dial Extensometer	0.0002	0.2	0.000 04	200	0.040
Berry Strain Gage	0.0002	0.2	0.000 04	300	0.060
Olsen New Extensometer (Catalog 10, Part A, p. 75)	0.001 0.0001 by Vernier	...	....	50	0.050

TABLE III.—CALIBRATION OF AMSLER-MARTENS EXTENSOMETER.

Calibrating Device			Amsler-Martens Extensometer							
Actual Reading, in.	Corrected Reading,[a] in.	Increment, in.	Scale No. 1		Scale No. 2		Average Increment, scale reading	Increment, in.[b]	Error, in.	Error, per cent
			Reading	Increment	Reading	Increment				
0.000 000	0.000 000	0.000 000	0.50	0.00	0.46	0.00	0.00	0.000 000	0.000 000	0.00
0.002 000	0.002 006	0.002 006	1.51	1.01	1.49	1.03	1.02	0.002 040	+0.000 034	+1.69
0.004 000	0.004 012	0.002 006	2.54	1.03	2.50	1.01	1.02	0.002 040	+0.000 034	+1.69
0.006 000	0.006 018	0.002 006	3.55	1.01	3.51	1.01	1.01	0.002 020	+0.000 014	+0.70
0.008 000	0.008 024	0.002 006	4.55	1.00	4.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.010 000	0.010 030	0.002 006	5.55	1.00	5.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.012 000	0.012 036	0.002 006	6.55	1.00	6.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.014 000	0.014 042	0.002 006	7.55	1.00	7.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.016 000	0.016 048	0.002 006	8.55	1.00	8.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.018 000	0.018 054	0.002 006	9.55	1.00	9.51	1.00	1.00	0.002 000	−0.000 006	−0.30
0.020 000	0.020 060	0.002 006	10.56	1.01	10.53	1.02	1.015	0.002 030	+0.000 024	+1.20
0.022 000	0.022 066	0.002 006	11.58	1.02	11.55	1.02	1.02	0.002 040	+0.000 034	+1.69

[a] Corrected Reading = 1.003 × Actual Reading.
[b] Increment in inches = Scale Reading ÷ 500.

The Ewing extensometer is provided with a micrometer screw for checking the instrument at frequent intervals in a very convenient way. One of these extensometers just after checking with its own micrometer screw was further checked using the extensometer calibrating device herein described. No observable errors were noted throughout the total range of the extensometer (140 divisions). It should be pointed out, however, that this particular extensometer had engraved divisions in its microscope that would not permit of estimating deformations closer than about ±0.00004 in.

The Amsler-Martens mirror extensometer is appreciably more sensitive than the Ewing but must be used throughout a rather limited total range if the desired accuracy of readings is to be maintained. This is shown by the data in Table III, wherein the errors as determined with the calibrating device are given for certain increments. Due consideration must be given to the limitations of this extensometer, especially when it is used for measuring the deformations of comparatively low modulus materials; otherwise appreciable errors will occur in the results obtained. Both the Amsler-Martens and the Ewing extensometers are quite free from backlash troubles, when properly used, making them well suited for elastic limit determinations and hysteresis measurements.

TABLE IV.—CALIBRATION OF RIEHLÉ DIAL EXTENSOMETER.

Calibrating Device		Riehlé Dial Extensometer			Error, per cent
Actual Reading, in.	Corrected Reading,[a] in.	Dial Reading	Deformation, in.	Error, in.	
0.000 00	0.000 00	0.0	0.000 00	0.000 00	0.00
0.000 20	0.000 20	0.7	0.000 14	−0.000 06	−30.00
0.000 40	0.000 40	1.6	0.000 32	−0.000 08	−20.00
0.000 60	0.000 60	2.7	0.000 54	−0.000 06	−10.00
0.000 80	0.000 80	3.8	0.000 76	−0.000 04	−5.00
0.001 00	0.001 00	4.8	0.000 96	−0.000 04	−4.00
0.001 20	0.001 20	5.9	0.001 18	−0.000 02	−1.67
0.001 40	0.001 40	6.9	0.001 38	−0.000 02	−1.43
0.001 60	0.001 60	7.9	0.001 58	−0.000 02	−1.25
0.001 80	0.001 81	8.9	0.001 78	−0.000 03	−1.66
0.002 00	0.002 01	9.9	0.001 98	−0.000 03	−1.49
0.002 20	0.002 21	10.8	0.002 16	−0.000 05	−2.26
0.002 40	0.002 41	11.8	0.002 36	−0.000 05	−2.07
0.002 60	0.002 61	12.9	0.002 58	−0.000 03	−1.15
0.002 80	0.002 81	14.0	0.002 80	−0.000 01	−0.36
0.003 00	0.003 01	15.0	0.003 00	−0.000 01	−0.33
0.003 20	0.003 21	15.9	0.003 18	−0.000 03	−0.93
0.003 40	0.003 41	16.9	0.003 38	−0.000 03	−0.88
0.003 60	0.003 61	18.0	0.003 60	−0.000 01	−0.28
0.003 80	0.003 81	19.0	0.003 80	−0.000 01	−0.26
0.004 00	0.004 01	20.0	0.004 00	−0.000 01	−0.25
0.006 00	0.006 02	30.0	0.006 00	−0.000 02	−0.33
0.008 00	0.008 02	40.0	0.008 00	−0.000 02	−0.25
0.010 00	0.010 03	50.0	0.010 00	−0.000 03	−0.30
0.012 00	0.012 04	59.8	0.011 96	−0.000 08	−0.66
0.014 00	0.014 04	69.9	0.013 98	−0.000 06	−0.43
0.016 00	0.016 05	79.4	0.015 88	−0.000 17	−1.06
0.018 00	0.018 05	89.3	0.017 86	−0.000 19	−1.05
0.020 00	0.020 06	99.7	0.019 94	−0.000 12	−0.60

[a] Corrected Reading = 1.003 × Actual Reading.

The more commonly used dial-type extensometers can be adjusted so as to give very satisfactory average deformation values, but it must be remembered that these instruments depend upon a dial gage for their accuracy. The dial gages generally used in such extensometers frequently have an error of one division at certain points in their range. This would correspond to a total deformation error of 0.0002 in. in an instrument with a 5-to-1 lever ratio. Experience with such extensometers has shown that better results are obtained if more determinations are made throughout a given deformation range than would be necessary with a more accurate instrument. This is shown by the calibration data given in Table IV where the errors in the individual readings are in some instances of quite serious magnitude while the average error may be considered negligible.

In conclusion, the author is of the opinion that commercial testing laboratories have not, in general, accorded the calibration of extensometers the consideration due this phase of materials testing. This, perhaps, has been due to the absence of suitable apparatus for carrying out the calibration work satisfactorily. The rather simple device described in this paper would appear to offer one solution of the problem.

DISCUSSION

Mr. Tuckerman.

MR. L. B. TUCKERMAN.[1]—Mr. Templin is to be congratulated in getting a very simple and yet apparently reliable method for making use of a calibrated screw for measuring small deformations. The use of plate fulcra insures that the troublesome variations in knife edges are not going to worry him. The knife edge is a very fine apparatus when it is maintained properly, but those who have dealt with knife-edge balances know the difficulty in the ordinary testing laboratory of maintaining knife edges under the conditions which exist there. The plate fulcrum is less determinate, but by making the dimensions large enough, you have a means of getting your ratios with all the accuracy which is necessary for the limited magnification involved in this device, and with the calibrated screw properly protected, the machine should be thoroughly reliable. I think you all see that it is simple and easy to use.

Mr. Moore.

THE PRESIDENT (*Mr. H. F. Moore*[2]).—The Chair cannot let this opportunity go by without expressing appreciation of the recognition that extensometers need to be calibrated. I believe that among the superstitions of the testing engineer of the past generation was the superstition that any test made with an extensometer was a precision test; we have yet to realize that extensometers need calibration as much as do testing machines.

Messrs. Gilkey and Vogt.

MESSRS. H. J. GILKEY[3] AND FREDRIK VOGT[4] (*by letter*).—In connection with the tests of models of arch dams being carried on at the University of Colorado by the U. S. Bureau of Reclamation and the Arch Dam Committee of Engineering Foundation, it has been necessary to devise convenient means for calibrating a variety of strain measuring equipment. Among the instruments needing calibration are some forty-five 0.0001-in. "Last Word" dials; about a dozen Ames dials, both 0.001 and 0.0001-in.; compressometers,[5] primarily for 6 by 12-in., 3 by 6-in. and 2 by 4-in. concrete and mortar cylinders; strain gages of the Olsen and Berry types, etc.

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[1] Engineer Physicist, U. S. Bureau of Standards, Chevy Chase, Md.

[2] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

[3] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.

[4] Of Trondhjem, Norway, temporarily employed by the U. S. Bureau of Reclamation for the investigation of arch dam models at Boulder, Colo.

[5] These are "home made" instruments of the averaging type similar to those used for years by the University of Illinois and Portland Cement Association laboratories and described in detail in *Bulletin 5*, p. 8, of the Structural Materials Research Laboratory. So far as is known this type of compressometer has not previously been used on specimens smaller than 6 by 12 in.

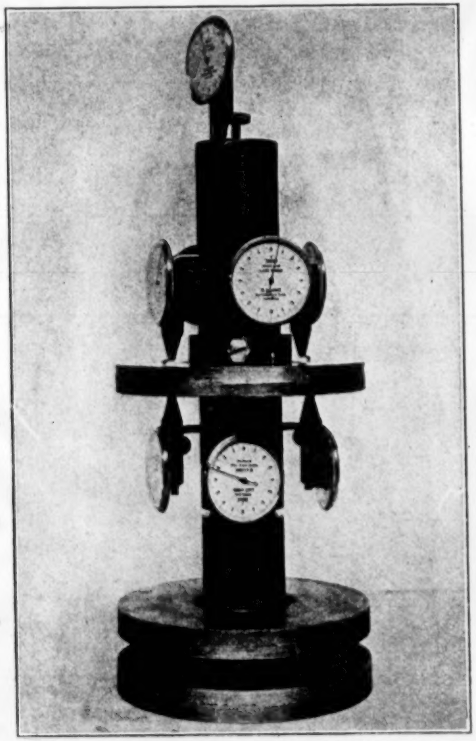

FIG. 1.—Showing the Simultaneous Calibration of Eight "Last Word" Dials.

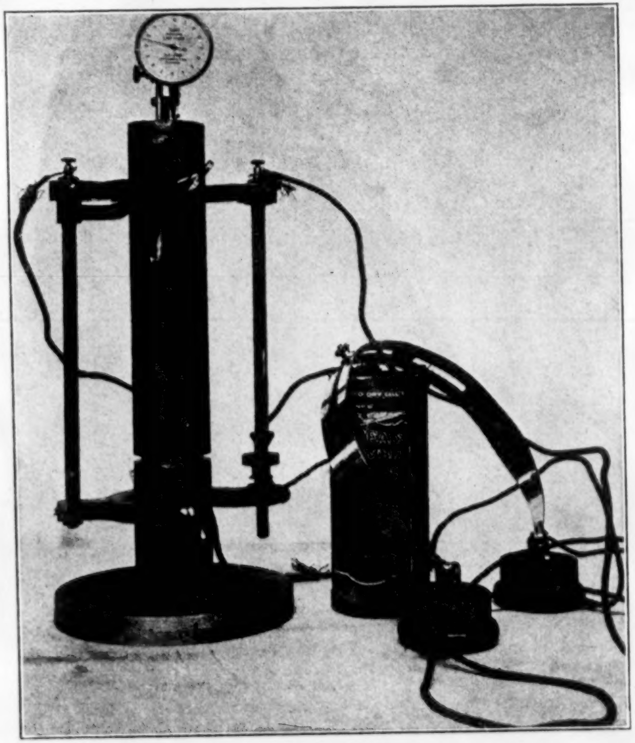

FIG. 2.—Showing the Calibration of a Riehlé Electrical Contact Micrometer Extensometer.

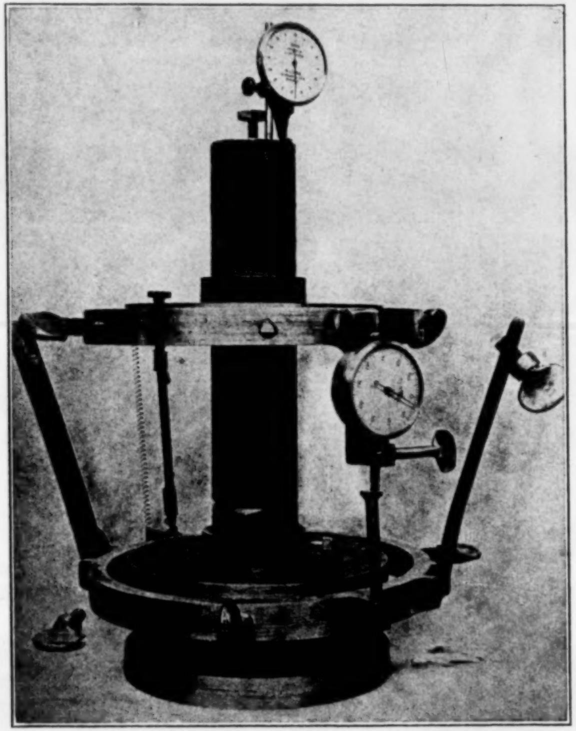

FIG. 3.—Showing the Calibration of One of the Home Made Averaging Type Compressometers.

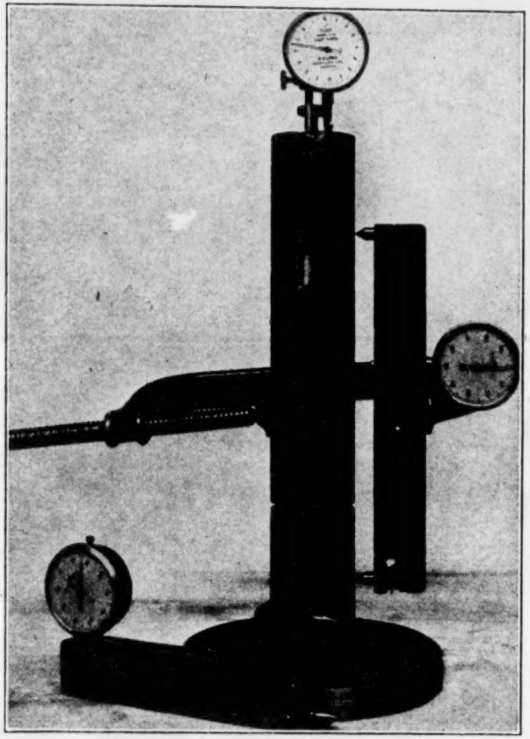

FIG. 4.—Showing a 0.0002-in. Berry Strain Gage Held in Position for Calibrating by a Carpenter's Clamp.

Messrs. Gilkey and Vogt.

Not so high a degree of refinement is necessary as is indicated by Mr. Templin for his instrument (total deformation of 0.000002 in.). The calibrator designed to meet the local need has proved to be entirely satisfactory and reliable to 0.00002 in. Just what the limit of attainable refinement may be is not yet know. Certainly the calibrator has demonstrated its adequacy for all the types of extensometers listed by Mr. Templin as well as its suitability for the compressometers, dials and other strain measuring devices.

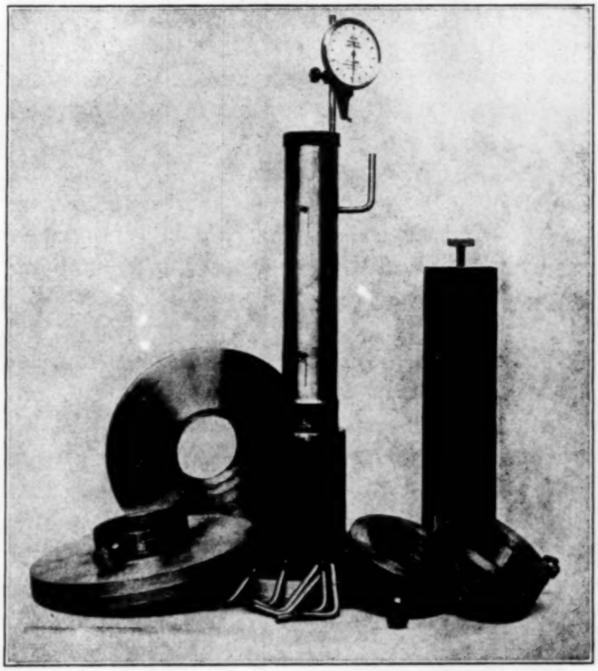

FIG. 5.—Showing the Dismantled Calibrator.

The writers are not offering the instrument that they have devised as either a better one or a substitute for that described by Mr. Templin. It is, however, a very simple and a very universally applicable calibrating mechanism and one that should meet the need of the great majority of laboratories.

The accompanying Figs. 1 to 5 illustrate the arrangement and use in a self explanatory manner. The instrument to be calibrated spans the crack near the lower portion. The upper movable sleeve is raised or lowered any desired amount by the milled screw at the top. As shown a 0.0001-in. "Last Word" dial previously calibrated

Messrs. Gilkey and Vogt.

by the manufacturer is used as the standard. Simultaneous readings are taken upon the dial and the instrument being calibrated for any desired interval. The dial divisions can be accurately estimated to 0.00002 in., that is, to the nearest fifth of a division. The movement is direct, there being no lever or multiplying system. The machine work on the shaft and the sleeve must be good. By lengthening the sleeve and the vertical shaft along which the sleeve slides, the possibility of lateral tilting as a source of error can be diminished to almost any point desired.

Many devices might be selected for the standard. A calibrated micrometer screw could be used at the top instead of a dial and would probably be preferable. An ordinary machinist's micrometer could be employed either as the standard or as the instrument to be standardized.

For calibrating dials (either the Ames or the "Last Word"), the dials are attached to small bars that screw through slotted holes in the sleeve into the fixed vertical shaft. The dial plungers bear against the collar that is placed on the movable sleeve and held by a set screw. Figure 1 shows the simultaneous calibration of eight "Last Word" dials.

Figure 2 shows a Riehlé electrical contact micrometer extensometer (0.0001 in.) in place for calibration.

Figure 3 illustrates the calibration of one of the home made averaging type compressometers mentioned earlier. Two collars of a diameter about equal to that of specimens for which the compressometer is designed are attached to the sleeve by set screws.

Figure 4 shows a 0.0002-in. Berry strain gage held in position for calibrating by a carpenter's clamp since the clamp furnished with the gage is too small for the sleeve.

Figure 5 shows the dismantled calibrator and should furnish all the information that is necessary for having one constructed with the exception of one necessary feature. A tapered screw should pass through a slot in the sleeve to the inner shaft in order to prevent any rotation of the sleeve on the shaft during calibration. The screw should preferably be of brass or other soft metal. The slot may be located almost anywhere. At the time the picture was taken this need had been met with a wooden wedge which was entirely satisfactory but less convenient than the screw. In designing such an instrument one should keep his own needs clearly in mind in order that the device may be of universal application so far as his laboratory is concerned. By a slight devising, the field of usefulness of such an instrument can be greatly broadened.

Messrs. Gilkey and Vogt.

In the course of preparing this discussion the writers looked up Mr. Templin's previous device mentioned on page 715. The calibrator that has just been described resembles Mr. Templin's instrument to a remarkable degree. The principles and operation are identical but the Colorado apparatus has been adapted to a somewhat broader field of use and is sufficiently heavy in itself to obviate the necessity of weights to maintain positive action. The Colorado instrument had been in service for weeks before anything was known about either of Mr. Templin's creations. It is very gratifying to find that our blind effort has accorded so perfectly with the solution reached by one of Mr. Templin's recognized standing in this field.

In spite of the similarity in principle and operation to that described by Mr. Templin in 1921[1] it is felt that the present device offers enough that is new by way of varied application and desirable structural differences to warrant presentation in this connection.

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[1] *Chemical and Metallurgical Engineering*, August 10, 1921, p. 248.

DETERMINATION OF CORRECT LOAD APPLICATION IN ROCKWELL HARDNESS TESTING MACHINES

By F. S. Mapes[1]

Synopsis

It being desirable to arrive at a simple and universal means of determining correct load application in Rockwell hardness testing machines, several methods were tried out to determine their simplicity and practicability. The final scheme adopted involved a steel bar supported on knife-edges with the load applied at the center. With a dial gage below, the load deflections of the bar could be accurately measured and by using this device in a standard vertical testing machine with loads of 10, 100 and 150 kg. applied, the bar was calibrated. The set-up was then transferred to the Rockwell hardness testing machine and check deflections secured.

By applying this method to three Rockwell machines it was found that very slight percentages of error were encountered. The method being simple, accurate and inexpensive, it is readily adaptable to commercial use.

Introduction

The usefulness of the Rockwell hardness testing machine depends almost entirely on the correct application to the specimen under test of, first, a minor load of 10 kg. and, second, a major load of 100 or 150 kg., depending on the material under test. Undoubtedly many users of this type of hardness tester have devised methods of determining that the proper loads are being applied. However, up to the present time there have been no generally acceptable or uniform methods of calibration. Probably the method most universally recognized is the use of standard bars of supposedly known hardness and uniformity. It is well known that this is only a comparative check and, with the many variables entering into the accuracy of this method, is very unsatisfactory. The problem, then, has been to devise a comparatively simple and yet adequately accurate method for determining that the loads are being properly applied.

Methods of Calibration

The first plan of calibration tried out by the author involved a system of levers and weights. A fulcrum was placed near the loading nose of the Rockwell hardness tester, and a beam resting under the loading nose at one end and supporting a weight at the other end was placed on this fulcrum. By applying dead weights we hoped to apply a counter pressure to the loading nose equal to the minor and major loads of the machine. This method was discarded because of its impracticability. The limits of travel of the loading nose were so small as to necessitate a complicated system of levers requiring accurate machining. Because of the expense thus involved and the infrequency of use for such a system, it was decided to aim at a more suitable method.

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[1] Testing Laboratory, General Electric Co., Schenectady, N. Y.

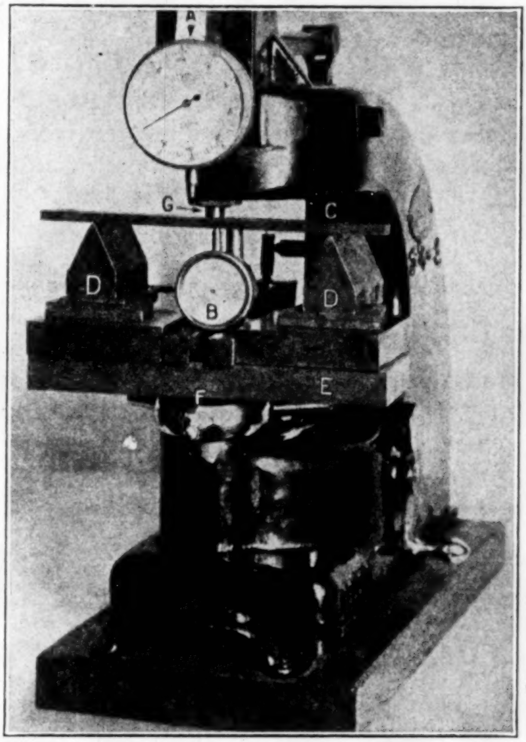

FIG. 1.—Method of Determining Load Application on a Rockwell Hardness Testing Machine.

A, Rockwell recording gage; *B*, dial gage for recording deflection of beam; *C*, calibrated beam; *D*, supporting knife edges for beam; *E*, steel supporting block; *F*, Rockwell elevating screw; *G*, Rockwell loading nose.

The second plan was a reversion to springs and screws. By calibrating the deflections of these in a standard vertical testing machine under loads of 10, 100 and 150 kg., it was hoped that check deflections could be obtained when the Rockwell loads were used. This scheme was also discarded due to the small limits of travel involved and because a large percentage of error was possible. A method in which a dial gage could be used was found to be the most desirable as the small limits of travel could then be measured. The final result was a plan which is believed to be simple, accurate, inexpensive and practical. In this, as well as in the previous methods, no impression is involved as neither the steel ball nor the diamond point are used.

As will be noted from Fig. 1, a steel supporting block, *E*, of sufficient strength and size to withstand any tendency toward bending under a load of 350 lb. was placed on the elevating screw, *F*, of the Rockwell machine. A piece of steel bar, *C*, was then selected so that it would give a maximum deflection of not greater than 0.020 in. when supported on knife edges as a simple, end-supported beam with a load of 150 kg. (approximately 331 lb.) applied at its center. This bar was set up in a vertical hydraulic testing machine having a capacity of 500 lb. and reading accurately to $\frac{1}{6}$ lb. Using an Ames dial gage for deflection, the bar was calibrated under loads corresponding in value to the minor and two major loads of the Rockwell hardness tester.

This calibrated piece of steel was then set up on the steel supporting block in the Rockwell machine with the knife edges spaced equi-distant from the loading nose. Below this bar and directly under the loading nose, an Ames dial gage, *B*, was placed to record any deflection at the center of the calibrated bar. Figure 1 shows in a self-explanatory manner the complete set-up used. The elevating screw was turned until bar *C* just came in contact with the loading nose. The minor and two major loads were then successively applied and deflections of the bar noted.

Test Data

The method of calibrating Rockwell hardness testers just described was applied to three machines owned by the General Electric Co., with the following results:

Loads, kg.	Deflections, in.			
	Testing Machine Calibration	Rockwell Machine No. 1	Rockwell Machine No. 2	Rockwell Machine No. 3
0	0	0	0	0
10	0.0012	0.0014	0.0012	0.0012
100	0.0051	0.0052	0.0050	0.0050
150	0.0076	0.0077	0.0074	0.0073

Conclusions

The results of these calibrations show a maximum percentage of variation of about 4 per cent which is a very satisfactory working variation, since it is practically impossible to secure such a close check by any other method.

A further justification of this method comes in its commercial application. The expense, trouble and time involved are small and the method does not require a technician for its successful application.

DISCUSSION

Messrs. Woodward and Rockwell.

Messrs. R. W. Woodward[1] and S. P. Rockwell[2] (*presented in written form*).—In the opening sentence of Mr. Mapes' paper, it is stated that "the usefulness of the Rockwell hardness testing machine depends entirely on the correct application" of minor and major loads. Actually, however, accuracy is more readily attained in the load application than in the other factors which comprise the test, and where inaccurate tests are obtained the load is seldom the cause of error. The Rockwell hardness tester is a complete measuring instrument, giving direct readings of hardness numerals on the Rockwell scale as differentiated from some hardness measuring systems (such as the Brinell), where two or more independent measuring machines are required to secure the final hardness values.

There are three major variables which enter into the Rockwell test. They are (1) the shape and size of the penetrator, (2) the loading mechanism, and (3) the depth measurement system. All of these have been standardized by the manufacturer of the instrument and a systematic calibration of the tester should include all of these variables.

The paper by Mr. Mapes describes a method for calibrating solely the load application, and leaves out of consideration the other two major variables which are equally important. His method is analogous to the use of proving rings, which has often been suggested for checking the load application of the Rockwell tester, and which although highly satisfactory for calibrating Brinell loads is open to criticism when applied to the Rockwell.

The penetrator head of the Rockwell moves only 0.008 in. for the softest materials tested on the "C" scale with 150-kg. load, and 0.010 in. for the "B" and "E" scales using 100-kg. load. This then requires that a calibrating beam such as described by Mr. Mapes should not deflect more than 0.008 in. with 150-kg. load in order that the motion of the penetrating load be kept within its working range. The stiff beam which these conditions call for increases the difficulty of its calibration and use.

The use of the usual type of 50,000 or 100,000-lb. testing machine found in most testing laboratories for calibrating the beam is also open to objection, since few if any of these machines can be operated

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[1] Consulting Metallurgist, The Stanley P. Rockwell Co., Hartford, Conn.

[2] President, The Stanley P. Rockwell Co., Hartford, Conn.

Messrs. Woodward and Rockwell.

with the proper degree of accuracy at loads as small as 22 lb. or even 330 lb. Dead loading or a low-range testing machine would be preferable.

In using the proving beam considerable care must be exercised that the plunger rod *G* shown in Fig. 1 of the paper makes absolutely flat contact with the beam *C*. Otherwise the rod *G* will be canted to one side and introduce friction, thus decreasing the applied load. Proving rings generally use ball contact, which overcomes this difficulty.

Bearing in mind all these considerations we believe it is quite possible that under the usual laboratory conditions the calibration and use of a proving beam will introduce larger errors than exist in the Rockwell hardness tester, which is in itself a precision machine.

Returning to the other variables which must be considered in a complete calibration of the Rockwell tester, we find that the ball penetrators can be readily checked with measuring instruments of suitable precision. The Brale, or diamond cone penetrator, cannot be so readily measured since both the angle of the cone and the curvature of the apex must be rigidly accurate and require projection equipment for proper inspection and measurement. The use of improper cones, or cones which have been redressed by local lapidaries, probably is the greatest source of error in the use of the Rockwell tester.

The depth measuring component consists of a dial indicator actuated by the penetrating head through a magnifying lever. This combination can be calibrated through fundamental standards, but when it is remembered that one point on the dial represents a motion of the head of 0.00008 in., it can be appreciated that few testing laboratories are equipped with calibrating devices of the required sensitivity and accuracy.

Considering, then, the requirements for a complete calibration of the Rockwell tester, it can be readily seen that it is quite an undertaking for most testing laboratories, and that in the final analysis resort must be made to standard test blocks. By using two standard test blocks of widely different hardness range for each hardness scale, a complete verification is assured. If only a single test block is used, compensating errors might exist which would give a correct reading at one end of the hardness scale, whereas if the opposite end of the scale were also checked, it is improbable that exact compensation could also exist.

Mr. Mapes contends that the use of standard blocks is very unsatisfactory, since these are of unknown hardness and uniformity. This statement probably is true where the user attempts to construct his own test blocks. The sensitivity of the Rockwell test is such that

Messrs. Woodward and Rockwell.

it readily detects non-uniformity in a block of metal, and it is extremely difficult to heat-treat a piece of steel or non-ferrous metal to be entirely homogeneous. In fact, there are a very limited number of heat-treating departments which have been able to produce test blocks of a uniformity satisfactory for distribution to users.

The test blocks which the manufacturer of the Rockwell tester supplies are all rigidly inspected for uniformity and individually calibrated on master testing machines, which in turn have had all the variables calibrated through fundamental standards. We believe, therefore, that the use of proper standard test blocks represents the simplest and most accurate method for the operator of the Rockwell hardness tester to verify the correctness of all the variables encountered in a complete hardness test.

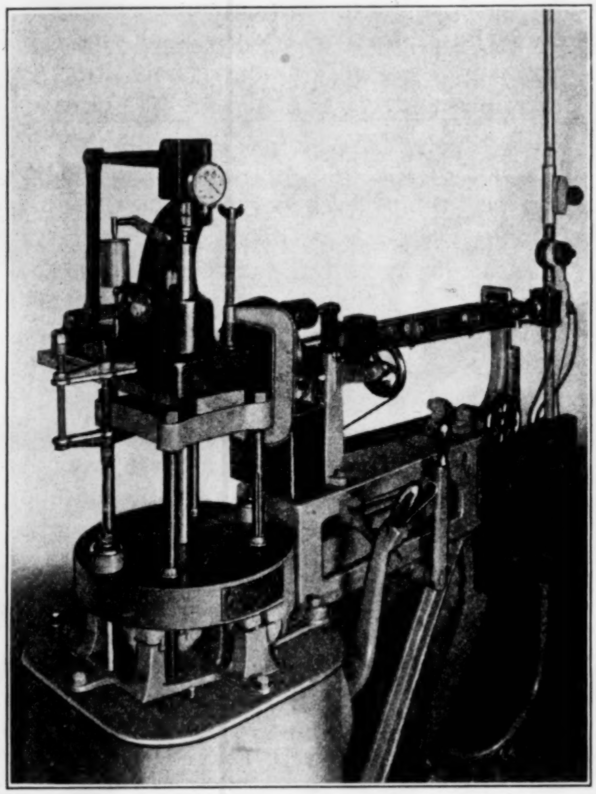

FIG. 1.—Illustrating Method of Weighing Load in Calibrating Rockwell Hardness Tester.

Mr. Townsend.

MR. J. R. TOWNSEND.[1]—Mr. Mapes' method of checking the loads applied by the Rockwell hardness tester has much to recommend it on the basis of simplicity, and the equipment necessary may usually be found available by a testing laboratory.

There are two comments I should like to make upon this paper. First, I wish to describe another method of checking the applied load on the Rockwell tester and, second, to discuss the calibration of the machine.

The method of checking the applied load on the Rockwell hardness tester described below has been in use in our laboratories for a period of over two years. It consists in directly measuring the load upon the penetrator of the Rockwell machine by employing the compression head of a universal testing machine as a weighing device. This is accomplished by removing the anvil supporting screw from the hardness tester and placing the machine upon a testing machine capable of determining the true load on the penetrator of the hardness tester to an accuracy of ± 0.5 per cent. This accuracy can be obtained on a properly adjusted and calibrated testing machine. The load is carried from the penetrator to the weighing table of the testing machine by a solid metal bar capable of supporting this load as a column without appreciable deformation. The "minor" load of 10 kg. is first determined by raising the pressure on the penetrator until the dial and index line reads zero and then weighing the resulting load. The "major" load is then applied in the usual manner, and weighed. The accompanying Fig. 1 illustrates the method of weighing the load. Provided the weights and levers are not subject to damage, one check should be sufficient for the life of the machine. It is also important to check the dial and the other working parts of the machine, and since these are affected by wear they should be more frequently checked.

With regard to calibration it has been our experience that the only satisfactory method of accomplishing this on the Rockwell hardness tester is with standard blocks, since this method evaluates all the diverse effects of friction, wear, centering, adjustment, etc., which are not apparent by checking the load as suggested. A review of the performance of a number of Rockwell testers under actual inspection conditions was given in a paper by Van Deusen, Shaw and Davis entitled, "Physical Properties and Methods of Test for Sheet Brass" presented before this Society last year.[2]

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[1] General Development Laboratory, Bell Telephone Laboratories, Inc., New York City.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 27, Part II, p. 173 (1927).

Mr. Mapes. MR. F. S. MAPES.[1]—Commenting upon the remarks made by Mr. Woodward, the purpose of this calibration was not to check all the major variables, as Mr. Woodward has pointed out. But in calibrating the load we have eliminated one possible source of error.

There is no reason why we cannot secure a beam which is stiff enough to register the deflection noted under the minor and major loads. Incidentally the bar which was used was picked at random from a junk pile and was probably only a nickel steel bar. It certainly went through no permanent deformation. In calibrating this bar, an Amsler machine was used with a 500-lb. scale, the capacity of the machine being 500 lb. The machine was accurate as checked by the manufacturer to $\frac{1}{6}$ lb., so I believe no error is introduced there.

In considering friction, which might be caused by the bar not being perfectly level, we used levels in lining up the machine on the setup itself, so as to eliminate this source of error.

The other method of calibrating this Rockwell machine as suggested by Mr. Townsend was tried out by us, but due to the fact that the machine we used was of large capacity we found that too many errors were noted, and it was unsatisfactory. I believe also that in a great number of laboratories or in the field, it would be practically impossible to find a machine which could be used for this purpose.

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[1] Testing Laboratory, General Electric Co., Schenectady, N. Y.

MEASUREMENT OF BRITTLENESS

By P. L. Irwin[1]

Synopsis

Tests of brittle materials usually yield results of larger variation or scatter than the condition of the material would seem to justify. The tension test has usually been employed, but the author contends that the flexure test, recommended by earlier investigators, is the better type of test from the theoretical as well as experimental standpoint.

Results of brittleness tests on thin spring steel, which has been embrittled by electrolytic cleaning, are given which show the effect of duration of treatment, current density, and time and temperature recuperation.

Introduction

Failure of steel due to hydrogen brittleness is frequently encountered in the manufacturing industries, usually as a result of cleaning or plating processes. Extensive researches on the failure of boiler drums and acid kettles have shown that hydrogen brittleness is often an acquired property due to the service to which the material is subjected. In any case, its presence is dangerous and costly.

Electrolytic cleaning and plating of motor brush-holder springs gave rise to hydrogen-brittleness failures in consequence of which the Research Department was requested to determine, if possible, the factors that influence such embrittlement. Similar failures due to acid pickling, and likewise caused by the production of nascent hydrogen, have been reported from time to time.

A review of the published data on brittleness tests revealed the fact that tension, bending, impact, and reversed flexure have been employed to measure brittleness with varying degrees of success.[2] A large amount of work has been done by means of the tension test, but due to the fact that materials in a brittle state are usually so due to a brittle case or at least to a decided difference in hardness between the center and extreme fibers, the test result is an average of the stress supported by the whole section, ductile and brittle, up to the point of rupture of the brittle material. After the crack starts in the brittle material, lateral constraint and stress concentration may cause simultaneous fracture of the ductile material in cases where the depth of case is large.[1] From this it will be seen that in a tension test the presence of a brittle exterior may be partly masked by the support of the core material. In the bending test, however, the extreme fibers are subjected to the maximum stress, which condition is, in our opinion, a better one for a brittleness test.

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[1] Mechanical Engineer, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa

[2] Bibliography No. S-3803, "Caustic or Hydrogen Embrittlement of Steel," Engineering Societies library. From 1887 to 1926.

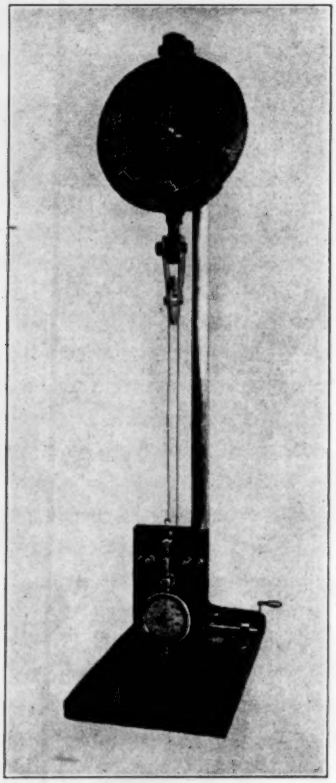

FIG. 1.—Brittleness Testing Machine.

PROCEDURE

It seemed very desirable, therefore, to devise a flexure test in which the following conditions would be satisfied:

1. Material tested to be that used for brush-holder springs.
2. Area of test piece should be large in proportion to the volume in order to obtain the maximum effect of hydrogen penetration.

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[1] Hillman and Clark, "Cyanide Brittleness," *Proceedings*, Am. Soc. Steel Treating, Vol. 10, No. 6, p. 954 (1926).

3. Uniform cross-section of test specimen to avoid stress concentration.
4. Simple stress application to give a commensurable stress value.

It was found that a simple flexure test on a piece of straight flat spring material, using a span of $1\frac{1}{2}$ in., satisfied all of these conditions.

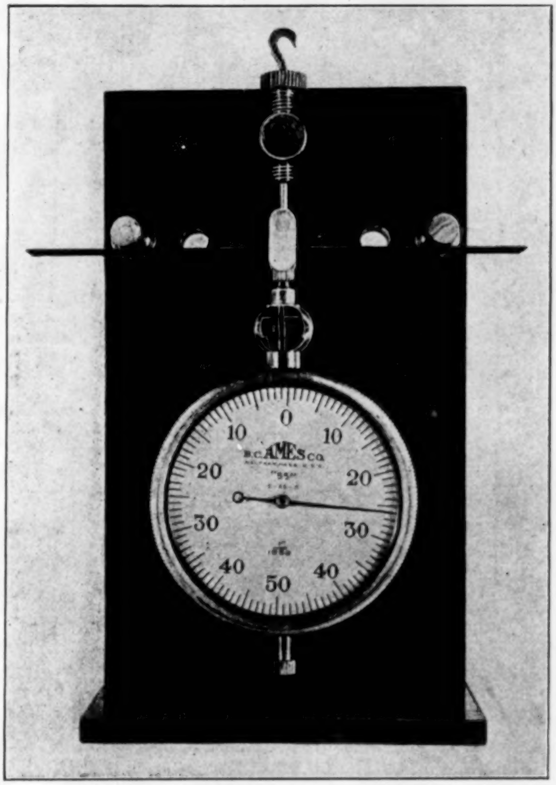

FIG. 2.—Close-up View of the Flexure Device

TESTING DEVICE

The next step was to devise a means of applying and measuring a load at the center of the span. This yields but one answer: ultimate load to fracture. By adding a means of measuring the deflection of the test specimen at the center of the span a load-deflection curve may be plotted. Such a curve gives the values of proportional limit and modulus of elasticity.

With these points in mind the machine shown in Fig. 1 was devised. By means of a worm-gear reduction, the crank turns a reel to which a violin E-string is attached. This string passes over the pulley on the weighing rod of the spring balance and is fastened to the stirrup which passes under the middle of the test specimen, Fig. 2. This stirrup is guided by a drilled machine screw which also serves as a dead-stop when the test specimen breaks. The dial gage is attached to the vertical plate in such a position that its stem presses against the under side of the stirrup, giving curves such as the one shown in Fig. 3.

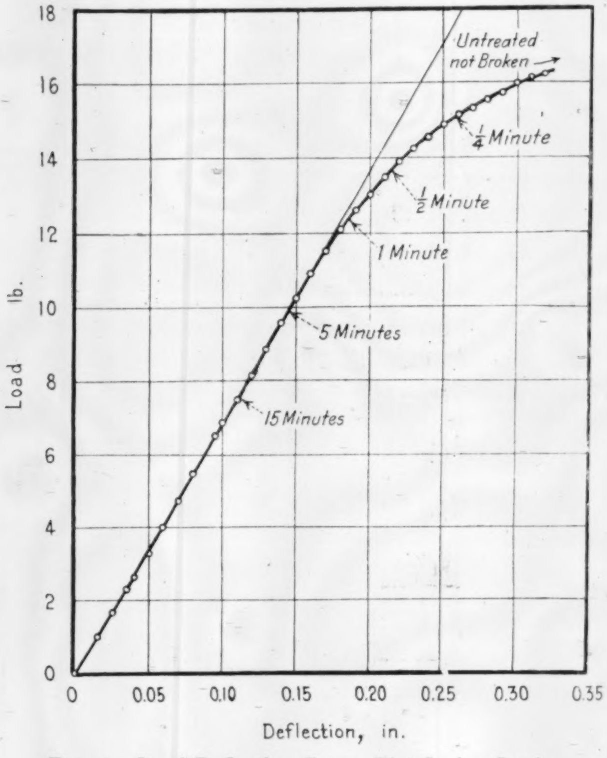

FIG. 3.—Load-Deflection Curve, Flat Spring Steel.

Specimen 0.245 in. Wide, 0.020 in. Thick.

TESTS

In order that a spring may be plated (or galvanized) it is necessary to remove the smooth blue oxide which forms during tempering. This is easily accomplished by making the spring the cathode in a sulfuric-acid bath for half a minute, more or less, depending upon the current density. If the spring is plated electrolytically, it is again made the cathode in the process. Therefore, it will be seen that either or both treatments may bring about brittleness of the spring.

The two principal variables in the treatment are current density and length of time of treatment.[1] Another factor of importance is the interval of time between the removal of the test specimen from the bath and the moment of fracture.

Three series of tests were made, in each of which two of the factors noted above were held constant and the third was made variable. In the first series, the current density and the interval between treatment and test were made constant, the time or duration of treatment being varied. Load and deflection readings were taken on each specimen and when plotted to the same scale were found to be identical with the load-deflection diagram of a normal (untreated) test specimen except for the fact that the treated pieces broke at a smaller load, depending upon the duration of treatment. Figure 3 shows the normal curve with the breaking points for tests of $\frac{1}{4}$, $\frac{1}{2}$, 1, 5, and 15 minutes duration of treatment. Since the curves were identical except for breaking strength, the data were plotted as shown in Fig. 4. For truly brittle pieces, we may take either the decrease in maximum deflection or the decrease in maximum load as a measure of the brittleness, since they are proportional. For less brittle pieces the graph (Fig. 3) curves away from the modulus line,

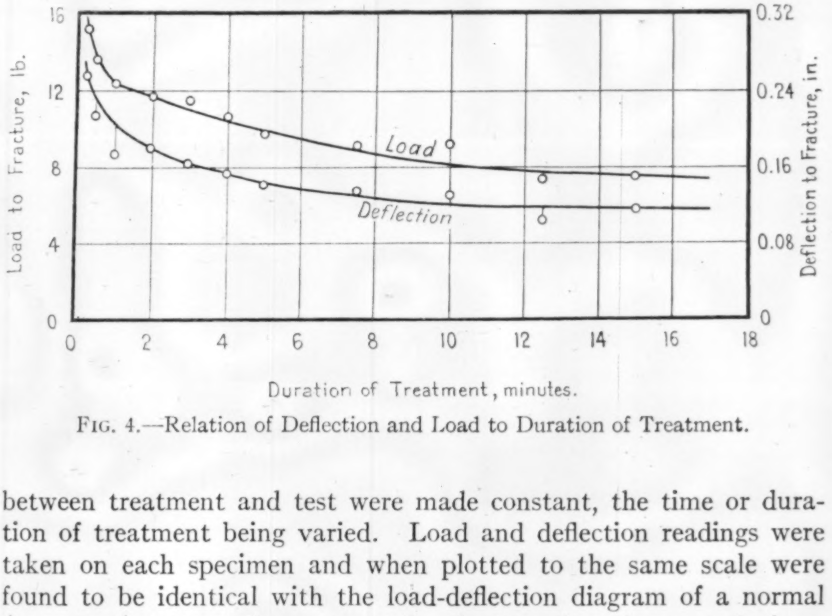

FIG. 4.—Relation of Deflection and Load to Duration of Treatment.

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[1] All tests were made at room temperature.

in which case it is more reasonable to take the decrease in deflection as a measure of brittleness. Both factors have been plotted for comparison, each point representing the average of at least two tests. The embrittlement is seen to increase rapidly with time of treatment during the first two minutes, the decrease in strength from this point becoming more gradual.

The tests of the second series were made under conditions of variable current density, the other factors being constant. Figure 5 brings out the marked decrease in strength up to a current density of 3.3 amperes per sq. in. With a further increase of current density the increase in brittleness drops off rapidly. This may be brought about, according to Mr. D. R. Kellogg,[1] by the rise in pressure of the hydrogen about the test specimen to a maximum and an accompanying hydrogen saturation of the test specimen, in the outer region at least.

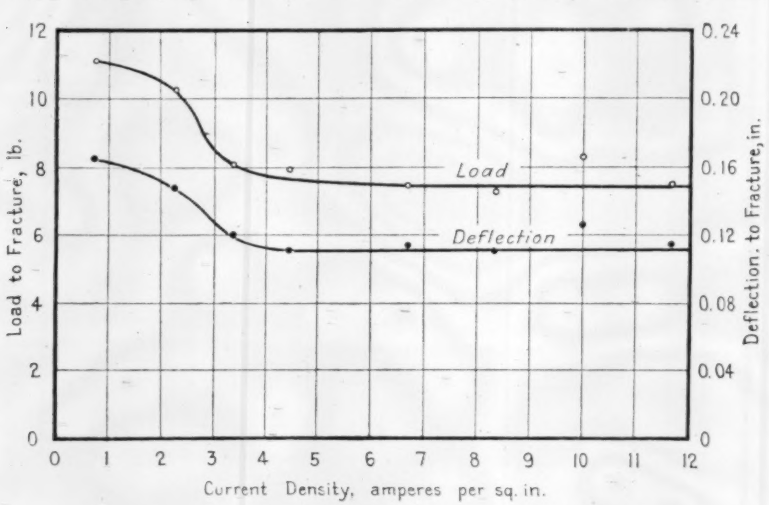

FIG. 5.—Relation of Deflection and Load to Current Density.

The third series of tests was made with all factors constant except the time interval between the removal of the test specimen from the bath and fracture. The aging is seen to have a marked recuperative effect during the first half hour, as shown in Fig. 6, which gives the relation of ultimate load to interval for a range of 2 to 80 minutes, beyond which the curve gradually rises to a value which would indicate complete recovery of the specimen in 5 days.

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[1] Section Engineer, Chemical Section, Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

Three test specimens were treated for 5 minutes at 10 amperes each and then placed in an electric oven at a temperature of 100° C. for 15 minutes. They were then tested and found to be normal in strength. Since the tests were made at 35 to 40 minutes after removal from the electrolyte, the ultimate load to fracture according to our recovery curve, based upon time alone, should have been 11.2 lb. All three bent double at 16.3 lb. without fracture, indicating rapid recovery by this method. This method has been published by Storey[1] and others and was tried here as an interesting check.

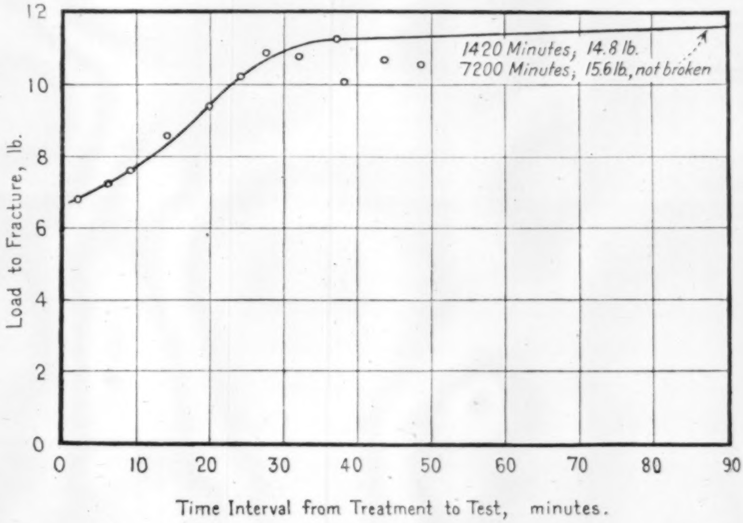

FIG. 6.—Recuperation Curve, Relation of Ultimate Load to Time Interval from Treatment to Test.

CONCLUSIONS

The use of the flexure test for brittle materials has the following advantages:

Strain of the extreme fibers is more easily measured than in the tension test, since the deflections are many times larger than the stretch of the extreme fiber.

The difficulties of the tension test, in gripping flat test specimens and in obtaining axial pull, are eliminated.

The careful machining and expense of fatigue or impact tests is unnecessary.

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[1] O. W. Storey, "A Microscopic Study of Electrolytic Iron," Am. Electrochemical Soc., Vol. 25, p. 489 (1914).

Brittle materials give stress-strain curves that are straight lines almost up to the point of fracture. Hence, for a truly brittle material the degree of brittleness may be taken as the percentage decrease in normal load or deflection required to cause fracture.[1] For materials which are more or less ductile in the treated condition, however, the decrease in the deflection required to cause fracture should be used. In case the untreated material cannot be broken on a reasonably short span, the normal load and deflection may be taken at a deflection of one-third the span or at the point of sudden yielding.

It is hoped that this paper will emphasize the advantages of the flexure test for brittle materials and serve to substantiate the claims of earlier investigators as to the value of that test for this purpose. We believe that the data given in this paper, as well as the method of measuring brittleness, will be of interest to spring manufacturers and others interested in hydrogen brittleness.

Acknowledgment.—The author is indebted to the Westinghouse Electric and Manufacturing Co. for permission to publish these results, to Mr. A. T. Krogh for information regarding the preparation of specimens and to Mr. T. F. Hengstenberg for assistance in the tests.

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[1] Cast iron gives a stress-strain diagram which curves gradually from the start with no sudden yielding and may, therefore, be included in this class of materials. See Bach-Baumann, "Elastizität und Festigkeit."

STUDIES OF METHODS OF MAKING DEVAL ABRASION TESTS OF AGGREGATES

By Stanton Walker[1]

Synopsis

Considerable interest has been displayed by a number of different investigators in carrying out studies of the Deval abrasion test with the view of developing a method suitable for application to different types of aggregates. Studies looking toward the development of a suitable test method require consideration of such factors as the number of revolutions of cylinder, weight of sample, number of balls in abrasive charge, grading of sample, and shape of aggregate particles. This paper gives comprehensive data on the effect of these factors on the wear in the Deval abrasion testing machine for five aggregates, different in type.

No recommendations are made as to the best procedure to follow in carrying out the test, due to lack of information which permits of correlating similar results on a great number of different aggregates with their behavior in use. The data furnish valuable information, however, on the interpretation of the tests, which is a necessary preliminary to developing an abrasion test method as a criterion of aggregate quality.

Conclusions are stated at the close of the paper.

Introduction

Abrasion tests of aggregates for concrete have received considerable study in an effort to determine the effect of resistance to abrasion on their concrete-making properties. Many engineers consider this factor to be of considerable importance, although the studies which have been carried out have generally failed to indicate any definite relationship between the resistance to abrasion of an aggregate and the resistance to abrasion or compressive strength of concrete; the evidence is not quite so definite with respect to the transverse and tensile strength of concrete. It is not within the scope of this paper to discuss the merits of the resistance to abrasion as a criterion of the quality of concrete aggregates. That subject is of sufficient importance to be dealt with separately. This paper will be confined to a discussion of the effect of variations in methods of making Deval abrasion tests on the amount of wear of the aggregate.

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[1] Director, Engineering and Research Division, National Sand and Gravel Association, Washington, D. C.

The standard Deval abrasion test was developed about 1870 in connection with macadam road construction in France.[1] This test soon found its way to the United States and became one of the standard test methods of the American Society for Testing Materials in 1908. Under the title "Standard Method of Test for Abrasion of Rock" (D 2 – 26)[2] it has been continued as a standard without essential change since that time. While the standard test is not adapted for use in connection with studies of concrete aggregates and was not developed with that purpose in view, it was carried over into specifications for concrete with the development of concrete highway construction and is now generally used as one of the measures of quality of crushed stone as a concrete aggregate.

The sample for the standard test is required to consist as nearly as possible of fifty pieces of coarsely broken stone and to weigh within 10 g. of 5000 g. It is generally understood, and some specifications require, that the sample be hand broken.[3] Obviously such a sample cannot be obtained readily from gravel aggregates and it became desirable to develop a modification of the standard test for use with gravel. Such a modification was first introduced in about 1914[4] by A. S. Rea of the Ohio State Highway Department. As a result of Rea's studies, the American Association of State Highway Officials adopted, in 1917, a modified test for gravel. The sample for this test was originally graded from $\frac{1}{2}$ to 2 in. (screens with circular openings) and was composed of 50 per cent by weight of material passing the 2-in. screen and retained on the 1-in. screen and 50 per cent passing the 1-in. screen and retained on the $\frac{1}{2}$-in. screen. A 5000-g. sample was used and it was run in a standard Deval abrasion machine for 10,000 revolutions with six $1\frac{7}{8}$-in. cast-iron balls added as an abrasive charge. The material finer than a $\frac{1}{16}$-in. mesh sieve after the test was used in calculating the percentage of wear. In 1920 the requirements of the American Association of State Highway Officials for the grading of the sample were revised somewhat. A 5000-g. sample, made up of 1250 g. of each of the following four sizes of material was used: 2 in. to $1\frac{1}{2}$ in., $1\frac{1}{2}$-in. to 1 in., 1 in. to $\frac{3}{4}$ in., $\frac{3}{4}$ in. to $\frac{1}{2}$ in. (circular openings). In 1928 the requirements for the grading of the sample were again revised to permit the optional use of four gradings, one of which is that given above. The other three gradings provide for tests of samples having smaller sizes of particles.[1]

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[1] Original work of Deval in developing abrasion test for crushed rock for use in macadam road construction; *Annalles des Ponts et Chaussées*, 1879; *Bulletin Ministere des Travaux Publique*, 1881.

[2] 1927 Book of A.S.T.M. Standards, Part II, p. 463.

[3] See "Method of Test for Abrasion of Broken Stone and Broken Slag," U. S. Department of Agriculture *Bulletin No. 1216 Revised*, Tentative Standard Methods of Sampling and Testing Highway Materials (1928).

[4] A. S. Rea, "Standard Abrasion Test for Gravel Employed by the Ohio State Highway Department," *Good Roads*, June 6, 1914.

In some of the early work of the Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill., the methods of making the abrasion test were studied. As a result of these preliminary investigations, a comprehensive series of tests was outlined in 1920 in which such factors as the number of revolutions of cylinder, weight of sample, number of balls in abrasive charge, grading of sample, and type of aggregate were studied. The purpose of these tests was to determine the importance of such variables in selecting a test method and to establish the relationship between results of abrasion tests of different types of materials.

The purpose of this paper is to describe the results of these tests, which were carried out at the Structural Materials Research Laboratory under the direction of Duff A. Abrams, through the cooperation of the Portland Cement Association and Lewis Institute. The author was closely associated with the tests in his capacity as Associate Engineer. Permission to use the data has been granted by F. R. McMillan, Director of Research of the Portland Cement Association.

OUTLINE OF TESTS AND TEST DATA

The important question in connection with a study of methods of making abrasion tests of aggregate is, of course, to determine what method furnishes results which may be used most satisfactorily as a criterion of the usefulness of the aggregate. While this investigation was not designed to answer that question, it does furnish valuable information on the interpretation of the tests, which is necessary as a preliminary step to its consideration.

As stated above, the time-honored "Standard" method cannot be used for tests of gravel because of the requirements for the sample. Neither is it well adapted to tests of such aggregates as crushed blast-furnace slag; and even in the case of crushed stone it is awkward, as a sample of ledge rock is required.

The modifications of the standard test to meet special conditions, which have been proposed, required consideration of such factors as measurement of the wear, number of revolutions of cylinder, weight of sample, amount of abrasive charge, grading of sample, shape of particles, etc. The investigation under discussion gives comprehensive information on the effect of these variables.

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[1] See "Method of Test for Abrasion of Gravel," U. S. Department of Agriculture *Bulletin No. 1216 Revised*, Tentative Standard Methods of Sampling and Testing Highway Materials, 1928; see also *Proceedings*, Am. Concrete Inst., 1928, and 1928 Report of the Society's Committee D-4 on Road and Paving Materials, *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 521 (1928).

The tests were made in a standard Deval abrasion machine, running at a rate of 33 r. p. m. Such a machine is illustrated in Fig. 1. The samples were graded according to predetermined gradings and complete sieve analyses were made after the wear test had been run. The results are reported in terms of the percentage of wear, based on the amount finer than the No. 14 sieve after the test, and also in terms of the difference in fineness modulus[1] before and after the test. The detailed sieve analyses are quite voluminous and are omitted.

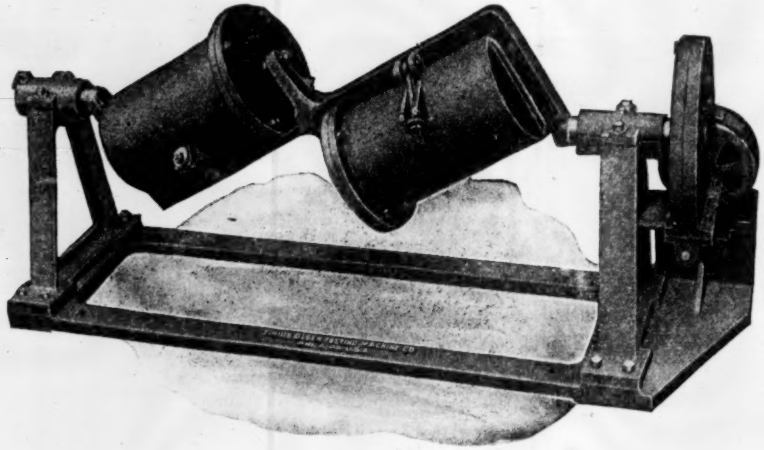

FIG. 1.—Deval Abrasion Testing Machine.

The following aggregates were used:

Gravel from Elgin, Ill.
Crushed Limestone from near Chicago, Ill.
Crushed Granite from Wisconsin.
Crushed Sandstone from Minnesota.
Crushed Blast Furnace Slag from the Chicago market.

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[1] The fineness modulus is the sum of the percentages in the sieve analysis, divided by 100, when the sieve analysis is reported in terms of percentages retained on sieves Nos. 100, 50, 30, 16, 8, and 4, ⅜ in., ¾ in., 1½ in., etc., conforming to the Society's Standard Specifications for Sieves for Testing Purposes (E 11 - 26). Tyler standard screen scale sieves No. 48, 28 and 14 conform to the requirements for sieves No. 50, 30 and 16, respectively.

TABLE I.—TYPICAL SIEVE ANALYSES OF SAMPLES AFTER TEST.

Kind of Aggregate	Weight of Sample, g.	Number of Revolutions	Number of Shot	Amounts Coarser than Each Sieve, per cent, by weight									Fineness Modulus	Difference in Fineness Modulus
				100	48	28	14	8	4	$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{2}$		
Grading of sample before test...........				100	100	100	100	100	100	50	50	0	7.00	....
Gravel.....	5000	1800	6	98.9	93.6	93.1	92.2	89.5	76.5	49.8	49.0	0	6.42	0.58
Gravel.....	5000	10000	6	93	67.4	66.5	65.6	63.7	58	49.3	47.8	0	5.11	1.89
Limestone..	5000	1800	6	98.6	95.5	94.4	92.7	89.0	74.3	49.4	46.7	0	6.40	0.60
Limestone..	5000	10000	6	94.7	75.0	72.3	69.8	64.8	53.6	47.5	44.4	0	5.22	1.78
Granite....	5000	1800	6	97.2	96.5	96.1	95.2	92.5	84.7	49.7	47.0	0	5.59	0.41
Granite....	5000	10000	6	85.0	80.7	78.2	76.4	72.1	65.6	49.3	47.1	0	5.54	1.46
Sandstone..	5000	1800	6	94.2	73.0	57.3	57.1	57.1	55.8	46.9	42.0	0	4.83	2.17
Sandstone..	5000	10000	6	85.4	54.8	43.0	43.0	43.0	43.0	42.8	39.0	0	3.94	3.06
Slag.......	5000	1800	6	96.9	96.0	95.1	93.9	91.4	83.2	48.3	45.5	0	6.50	0.50
Slag.......	5000	10000	6	89.5	87.1	84.7	82.0	77.3	64.8	46.3	40.2	0	5.72	1.28
Grading of sample before test...........				100	100	100	100	100	100	100	50	0	7.50	....
Gravel.....	5000	1800	6	98.9	98.7	98.7	98.6	98.4	97.6	92.3	47.6	0	7.31	0.19
Gravel.....	5000	10000	6	96.1	91.7	91.5	91.3	90.7	89.3	83.3	44.5	0	6.78	0.72
Limestone..	5000	1800	6	97.3	96.8	96.7	96.4	95.5	93	83	46.6	0	7.05	0.45
Limestone..	5000	10000	6	99.3	95.5	86.0	85.1	83.3	78.6	69.5	45.3	0	6.43	1.07
Granite....	5000	1800	6	98.0	97.9	97.7	97.6	96.8	94.9	85.9	46.9	0	7.16	0.34
Granite....	5000	10000	6	96.5	87.6	87.4	87.1	86.2	83.8	74.1	44.6	0	6.47	1.03
Sandstone..	5000	2000	6	92.9	86.1	82.3	82.2	82.2	82.0	76.8	38.5	0	6.23	1.27
Sandstone..	5000	10000	6	77.1	60.6	57.8	57.8	57.8	57.8	56.5	33.1	0	4.59	2.91
Slag.......	5000	1800	6	96.1	95.8	95.5	95.3	95.0	94.1	90.0	47.0	0	7.09	0.41
Slag.......	5000	10000	6	86.2	84.3	82.8	81.3	78.6	72.9	51.4	39.1	0	5.77	1.73
Grading of sample before test...........				100	100	100	100	100	100	75	25	0	7.00	....
Limestone..	5000	1800	6	97.9	96.5	96.1	95.2	92.9	83.2	62.8	21.3	0	6.46	0.54
Limestone.	5000	10000	6	91.1	82.4	80.9	78.6	74.1	63.2	47.9	21.6	0	5.40	1.60
Grading of sample before test...........				100	100	100	100	100	100	50	50	0	7.00	....
Gravel.....	2000	1800	6	80.9	77.9	76.5	75	71.9	62.8	48.3	47.2	0	5.40	1.60
Gravel.....	7000	1800	6	99.0	95.1	94.8	94.2	92.1	82.6	49.6	49.0	0	6.56	0.44
Limestone..	2000	1800	6	94.9	86.6	84.8	82.6	77.8	66.0	48.2	45.6	0	5.86	1.14
Limestone..	7000	1800	6	98.9	97.4	96.3	95.3	92.8	82.5	49.5	48.8	0	6.62	0.38
Slag.......	2000	1800	6	92.2	89.9	87.7	85.6	81.3	72.0	48.2	47.8	0	6.05	0.95
Slag.......	7000	1800	6	97.3	96.6	96.0	95.2	93.0	86.4	49.0	45.4	0	6.59	0.41
Gravel.....	5000	1800	0	99.5	99.3	99.2	99.0	98.5	94.6	50.6	49.9	0	6.91	0.09
Gravel.....	5000	1800	15	86.5	82.3	80.9	80.0	76.4	65.9	49.4	48.9	0	5.70	1.30
Limestone..	5000	1800	0	99.4	98.8	98.4	98.0	96.8	91.4	50.9	49.7	0	6.83	0.17
Limestone..	5000	1800	15	93.3	88.4	86.3	83.8	78.6	65.2	48.3	45.4	0	5.89	1.11
Granite....	5000	1800	0	99.4	99.4	99.2	99.0	98.1	95.5	51.7	49.3	0	6.92	0.08
Granite....	5000	1800	15	90.1	88.7	87.4	85.7	81.7	72.2	49.0	47.7	0	6.02	0.98
Sandstone..	5000	1800	0	96.8	90.3	85.4	85.3	85.2	82.1	48.5	44.3	0	6.18	0.82
Sandstone..	5000	1800	15	89.0	61.2	45.7	45.7	45.4	45.4	44.3	37.4	0	4.14	2.86
Slag.......	5000	1800	0	98.4	98.1	97.9	97.5	96.9	94.0	49.7	48.2	0	6.81	0.19
Slag.......	5000	1800	15	92.6	90.2	87.7	85.0	79.8	69.1	47.3	45.4	0	5.97	1.03

The test data are given in Tables I to VI and in Figs. 2 to 9, inclusive.

Five groups of tests were carried out in which the following variables were studied:

1. *Effect of Number of Revolutions of Cylinder.*—Tests made for revolutions ranging from 1000 to 10,000. Five aggregates, different in type, and three gradings of sample were used. The tests were made with a 5000-g. sample and an abrasive charge of six $1\frac{7}{8}$-in. cast-iron balls. See Table II and Figs. 2 and 3.

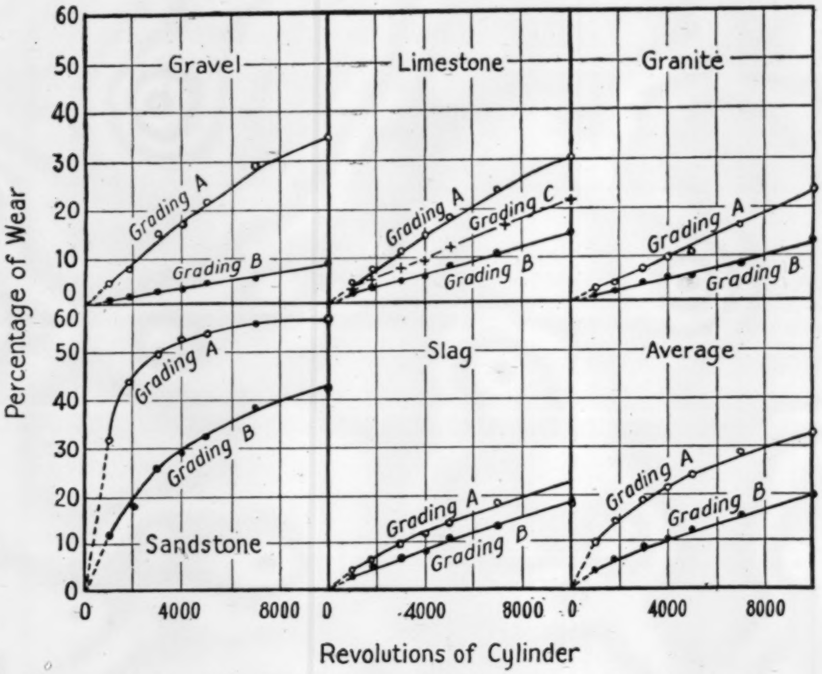

FIG. 2.—Effect of Number of Revolutions of Cylinder on Per Cent of Wear.
Data from Table II.
Compare Fig. 3.

2. *Effect of Weight of Sample.*—Weight of sample varied from 2000 to 7000 g. The tests were made with three aggregates, different in type, and with one grading. The number of revolutions was 1800 and the abrasive charge was six $1\frac{7}{8}$-in. cast-iron balls. See Table III and Figs. 4 and 5.

3. *Effect of Number of Cast-Iron Balls in Abrasive Charge.*—The number of $1\frac{7}{8}$-in. cast-iron balls was varied from none to 15. Five

aggregates, different in type, and two gradings were used. Tests were made for 1800 and 10,000 revolutions, using 5000-g. samples. See Table IV and Figs. 6 and 7.

4. *Effect of Grading of Sample.*—Fifty-seven different gradings, consisting of different proportions of the following three sizes of materials, were tested: No. 4 to $\frac{3}{8}$ in., $\frac{3}{8}$ in. to $\frac{3}{4}$ in., and $\frac{3}{4}$ in. to 1$\frac{1}{2}$ in. (square mesh sieves). Two aggregates, different in type, were tested.

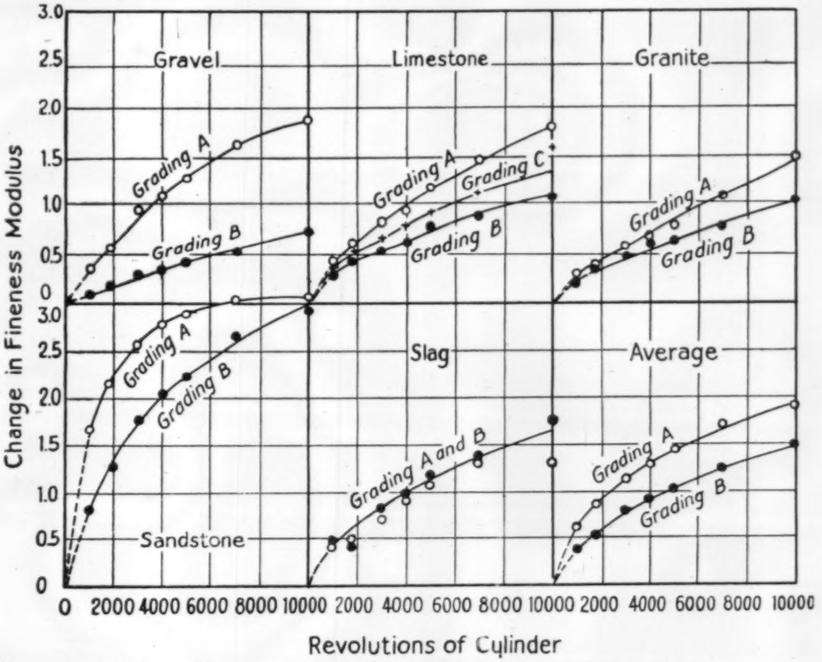

FIG. 3.—Effect of Number of Revolutions of Cylinder on Difference in Fineness Modulus.

Data from Table II.
Compare Fig. 2.

Samples of 5000 g. were run for 1800 revolutions, using an abrasive charge of six 1$\frac{7}{8}$-in. cast-iron balls. See Table V and Figs. 8 and 9.

5. *Effect of Shape of Particle.*[1]—Tests were carried out to study the wear of five different types of aggregates after the crushed particles had been worn to rounded form. These tests were made using samples consisting of fifty pieces and weighing 5000 g.[2] The tests were first run on four samples and then repeated, using the same

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[1] The results of these tests are reported in a discussion by Duff A. Abrams, *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 370 (1922).

[2] See the Standard Method of Test for Abrasion of Rock (D 2 – 26), 1927 Book A.S.T.M. Standards, Part II, p. 463.

material, for as many times as samples weighing 5000 g. and consisting of fifty pieces each could be obtained from the tested material. The machine was run for 10,000 revolutions; no abrasive charge was added. See Table VI and the photographs of the samples of aggregates before and after test in Fig. 10.

TABLE II.—EFFECT OF NUMBER OF REVOLUTIONS OF CYLINDER ON ABRASION OF AGGREGATE.

Weight of sample, 5000 g.
Abrasive charge, six $1\frac{7}{8}$ in. cast-iron balls.
Grading of sample:
A, 50 per cent No. 4 to $\frac{3}{8}$ in. and 50 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus, 7.00.
B, 50 per cent $\frac{3}{8}$ to $\frac{3}{4}$ in. and 50 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus 7.50.
C, 25 per cent No. 4 to $\frac{3}{8}$ in., 50 per cent $\frac{3}{8}$ to $\frac{3}{4}$ in. and 25 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus 7.00.

Number of Revolutions	Abrasion of Aggregate											
	Change in Fineness Modulus						Wear, per cent by weight finer than No. 14 sieve					
	Gravel	Limestone	Granite	Sandstone	Slag	Average	Gravel	Limestone	Granite	Sandstone	Slag	Average
GRADING A												
1000.....	0 35	0.41	0 27	1.66	0.41	0.62	4.2	4.3	3.0	31.7	4.1	9.4
1800.....	0.58	0 60	0.38	2.15	0 50	0.84	7.8	7.2	4.2	43.9	6.1	13.8
3000.....	0 96	0 83	0.59	2.56	0.70	1.13	14.6	11.0	7.4	49.7	9.1	18.4
4000.....	1.06	0.94	0.68	2.79	0 90	1.27	16.9	14.1	9.5	52.7	11.8	21.0
5000.....	1.28	1 17	0 78	2.88	1.03	1.43	21.3	18.0	10.9	53.7	13.9	23.6
7000.....	1 63	1 47	1 08	3.04	1.28	1.70	28.7	23.7	16.1	56.1	18.1	28.5
10000.....	1.89	1.78	1.48	3.06	1.28	1.90	34.4	30.2	23.6	57.0	18.0	32.6
Average...	1.11	1.03	0.75	2.59	0.87	1.27	18.2	15.5	10.7	49.3	11.6	21.0
GRADING B												
1000.....	0 10	0.28	0 23	0.83	0.47	0.38	0.9	2.2	1.5	11.5	2.8	3.8
1800.....	0 19	0 44	0 34	1.27[a]	0.41	0.53	1.4	3.6	2.4	17.8[a]	4.7	6.0
3000.....	0 29	0.53	0.49	1.77	0.82	0.78	2.6	4.8	4.2	25.6	6.3	8.7
4000.....	0.32	0 62	0 60	2.04	0.96	0 91	3.0	5.7	5.6	28.7	7.9	10.2
5000.....	0 42	0.76	0 61	2.22	1 18	1.04	4.2	8.2	5.9	32.4	10.5	12.2
7000.....	0 53	0.88	0.79	2.65	1.36	1.24	5.3	10 7	8.5	38.4	13.3	15.2
10000.....	0.72	1.07	1.03	2.91	1.75	1.50	8.7	14.9	12.9	42.2	18.7	19.5
Average...	0.37	0.63	0.59	1.96	1.00	0.91	3.8	7.1	5.9	28.1	9.2	10.8
GRADING C												
1000.....	....	0 36	....	....	....	....	....	3.4	....	....	....	....
1800.....	....	0 54	....	....	....	....	....	4.8	....	....	....	....
3000.....	....	0 70	....	....	....	....	....	8.0	....	....	....	....
4000.....	....	0.78	....	....	....	....	....	9.5	....	....	....	....
5000.....	....	0.92	....	....	....	....	....	12.2	....	....	....	....
7000.....	....	1 10	....	....	....	....	....	15.9	....	....	....	....
10000.....	....	1.60	....	....	....	....	....	21.4	....	....	....	....
Average...	....	0.86	....	....	....	....	....	10.8	....	....	....	....

[a] 2000 revolutions.

DISCUSSION OF TEST DATA

A careful study of the test results furnishes accurate information on the effects of the several variables studied and provides data essential in the selection of a suitable test method. The data do not, however, point to a particular procedure as being better than another, due to lack of information which permits of correlating these results with the behavior of the aggregates in service. Data of concrete tests are available on the aggregates included in this investigation,[1] but it is not considered that a large enough number of aggregates or a sufficient range in quality has been tested to permit of drawing definite conclusions along these lines. So far as these limited data go, no definite relationship between the resistance to abrasion of the aggregate and the concrete-making properties, as measured by wear, compressive strength and transverse strength, is shown. This discussion will be restricted, therefore, to a consideration of the effect of the individual variables on the results obtained in modified Deval abrasion tests.

TABLE III.—THE EFFECT OF WEIGHT OF SAMPLE ON ABRASION OF AGGREGATES.

Revolutions of cylinder, 1800.
Abrasive charge, six $1\frac{7}{8}$-in. cast-iron balls.
Grading of sample:
A, 50 per cent No. 4 to $\frac{3}{8}$ in. and 50 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus 7.00.

Weight of Sample, g.	Abrasion of Aggregate							
	Change in Fineness Modulus				Wear, per cent by weight finer than No. 14 sieve			
	Gravel	Limestone	Slag	Average	Gravel	Limestone	Slag	Average
2000	1.60	1.14	0.95	1.23	25.0	17.4	14.4	18.9
3000	1.13	0.80	0.72	0.88	16.8	11.4	9.3	12.5
4000	0.79	0.68	0.54	0.67	11.5	8.8	7.2	9.2
4500	0.69	0.59	0.51	0.60	9.9	8.0	6.6	8.2
5000	0.58	0.60	0.50	0.55	7.8	7.2	6.1	7.4
5500	0.50	0.56	0.53	0.53	7.0	7.6	6.8	7.1
6000	0.46	0.48	0.43	0.46	6.3	6.2	5.2	5.9
7000	0.44	0.38	0.41	0.41	5.8	4.7	4.8	5.1
Average	0.77	0.65	0.58	0.67	11.3	8.9	7.6	9.3

Method of Measuring Wear.—A complete sieve analysis was made of the sample after the test, using square mesh, wire-woven sieves of the Tyler standard screen scale. Table I shows typical sieve analyses of a few samples after the test which will serve to illustrate the nature of the detailed results. Only the percentage of wear, based on the amount passing the No. 14 sieve after the test, is reported for all of the samples. The No. 14 sieve conforms approximately to the size selected for the standard test and was used for that reason. In addition to the percentage of wear, differences in fineness modulus (before and after test) are also reported. The fineness modulus provides a convenient method for taking into account the amounts passing (or retained on) all of the sieves used in making the sieve analyses.

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[1] See *Bulletins 10* and *11* of the Structural Materials Research Laboratory.

A study of the percentages passing the Nos. 48, 28, 14 and 8 sieves indicate that there is little choice in sieve size within the range of those which would be selected as a measure of the dust resulting from abrasion. These are the only sieves which would seem to serve this purpose; sieves finer than about Nos. 48 or 28 differentiate between the fineness of the dust from different aggregates, while sieves coarser than about No. 8 take into account some of the smaller particles which have not been ground to dust. This latter statement is particularly true of a sample graded from the No. 4 sieve before the test.

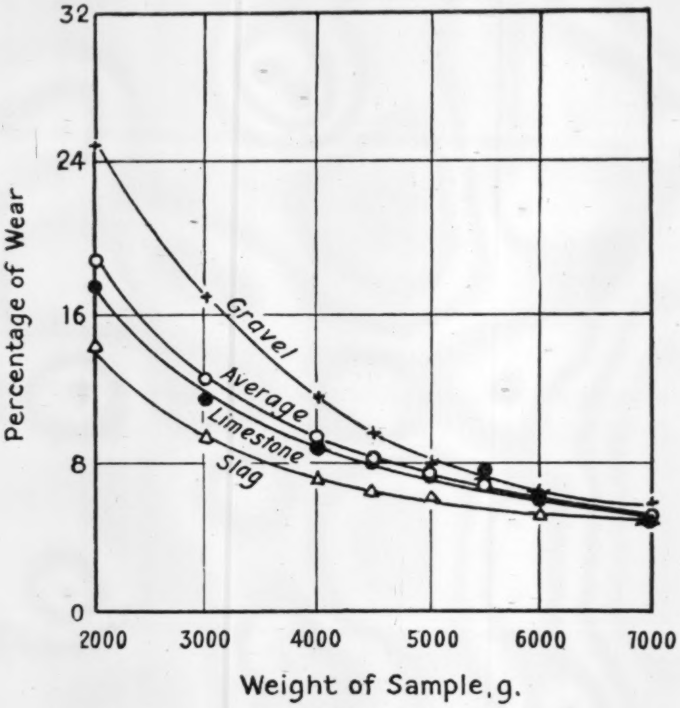

FIG. 4.—Effect of Weight of Sample on Percentage of Wear.
Data from Table III.
Compare Fig. 5.

There is sufficient difference in the percentages passing the Nos. 48, 28, 14 and 8 sieves, however, to indicate the desirability of taking into account more than one sieve in measuring the wear. Studies of methods of measuring abrasion suggested by the test data, such as percentages passing the various sieves, reduction of diameter of the various sizes, changes in the characteristics of sieve analyses curves, etc., lead to an almost endless number of possibilities, for which there is neither time nor space to discuss. The fineness modulus, however, takes into some account all of the various factors mentioned. Comparisons of the curves plotted for percentage of wear and difference in fineness modulus (see, for example, Figs. 2 and 3) show that very nearly the same relationships were obtained for both measures. A curve plotted between the two measures gives an approximately straight-line relation for a given group of tests. However, different curves are obtained for the different groups. Perhaps with the development of further information on the Deval test a more complate study of the different methods of measuring the abrasion will become important.

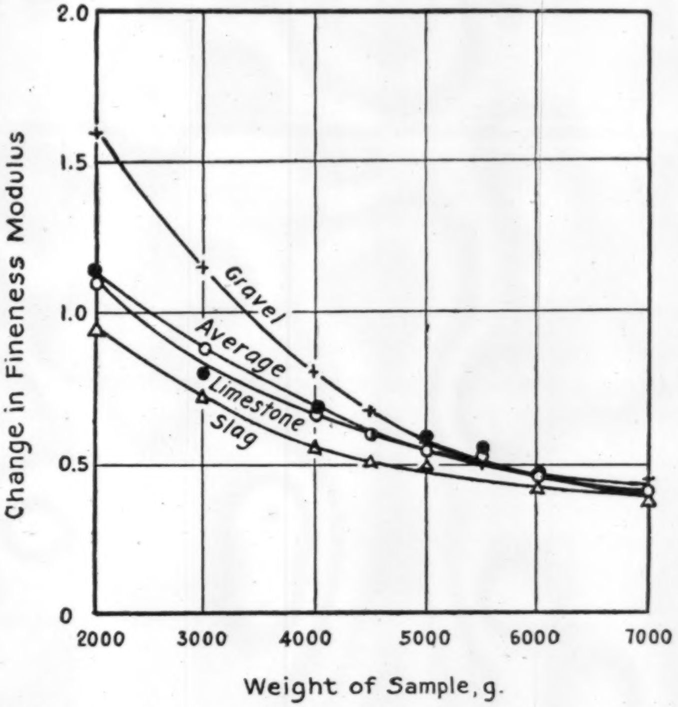

FIG. 5.—Effect of Weight of Sample on Difference in Fineness Modulus.

Data from Table III.
Compare Fig. 4.

Effect of Number of Revolutions of Cylinders.—Data showing the effect of number of revolutions of the Deval cylinder for a 5000-g. sample, with six $1\frac{7}{8}$-in. cast-iron balls added as an abrasive charge, are shown in Table II and in Figs. 2 and 3. Tests were made with five aggregates and two gradings, except for crushed limestone for which three gradings were tested. A study of the data shows that the type of aggregate played an important part in fixing the relationship between the number of revolutions and the amount of wear. For the harder materials the plotted points fall approximately on a straight line, showing that the amount of abrasion is proportional to the period of abrasion. For the softer materials and for the longer testing periods (larger number of revolutions) the line representing the points falls in a curve, which indicates a cushioning effect of the dust of abrasion.[1]

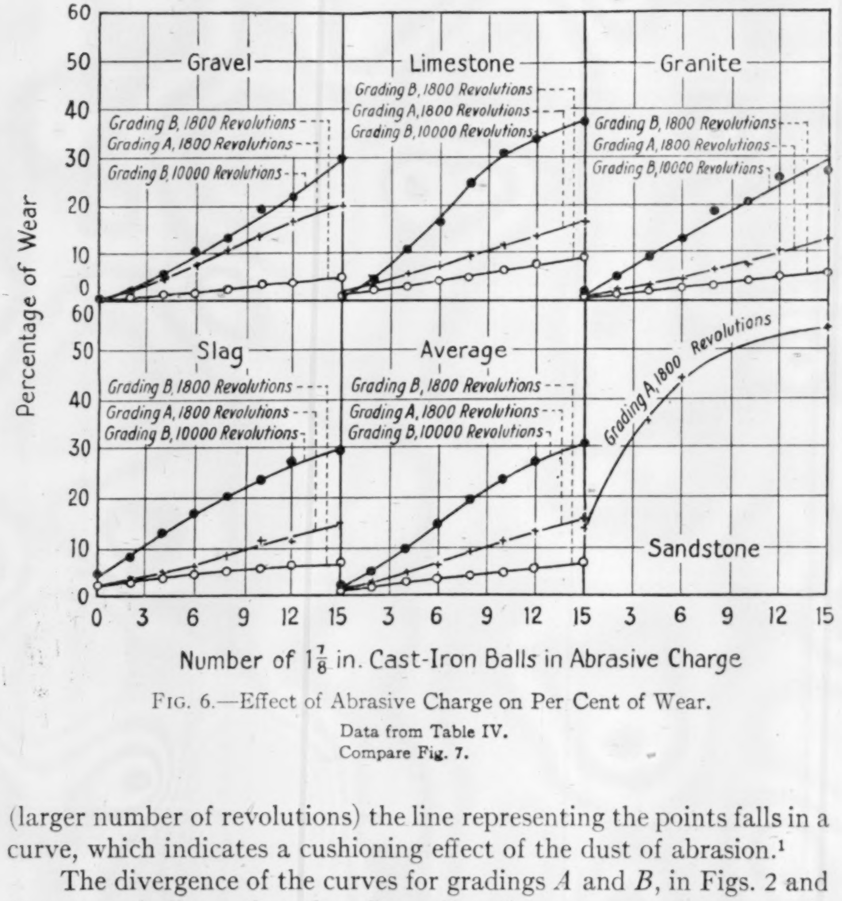

FIG. 6.—Effect of Abrasive Charge on Per Cent of Wear.

Data from Table IV.
Compare Fig. 7.

The divergence of the curves for gradings *A* and *B*, in Figs. 2 and 3, seem to indicate that the effect of grading was more important in the case of the gravel than in the case of the other aggregates tested. It should also be noted that the comparisons of the different materials were affected by the grading; for example, in the case of the gravel and the crushed limestone, the gravel showed a higher percentage of wear than the limestone for grading *A*, while the reverse is true for grading *B*. More wear was obtained for grading *A* than for grading *B* with all aggregates.

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[1] Several different investigators have studied the Deval test, using a slotted cylinder to eliminate the dust of abrasion. See "Use of Slotted Cylinder for Deval Abrasion Test of Rocks," report, N. Y. Commissioner of Highways, 1917. See also Report of State Commissioner of Highways, state of New York, 1917, results of tests on sandstone, limestone, slag, and gravel, using standard cylinder and slotted cylinder.

The data in Table II and Figs. 2 and 3 do not offer a definite indication as to the most satisfactory number of revolutions to use in the test. This factor is of principal interest in fixing the time required for the test. From the evidence available it would seem that to obtain dependable results with a 5000-g. sample and six balls as an abrasive charge, a number of revolutions of 2000 or greater should be used. Further data are required, however, to determine if a fewer number of revolutions will give satisfactory results using a smaller sample or larger abrasive charge.

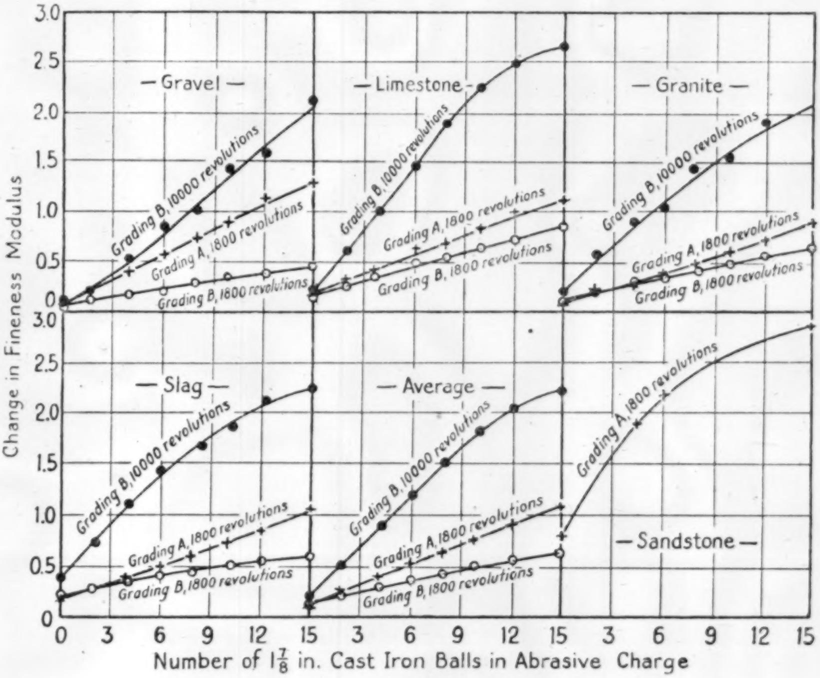


FIG. 7.—Effect of Abrasive Charge on Difference in Fineness Modulus.

Data from Table IV.

Compare Fig. 6.

Effect of Weight of Sample.—The data on the effect of the weight of sample are given in Table III and in Figs. 4 and 5. These results indicate that the weight of sample has an important effect on the amount of abrasion and on the relationship of the results for the different types of aggregate. For the three aggregates tested in this group (gravel, limestone and slag), the percentage of wear for the 2000-g. sample varied from 14 to 25, while for the 7000-g. sample the variation was only from 4.7 to 5.7. It seems evident from these tests that the size of the sample must be given careful consideration if a proper range in values is to be obtained. Study of the curves in

TABLE IV.—THE EFFECT OF ABRASIVE CHARGE ON ABRASION OF AGGREGATE.

Weight of sample, 5000 g.
Abrasive charge made up of $1\frac{7}{8}$-in. cast-iron balls.
Grading of sample:
A, 50 per cent No. 4 to $\frac{3}{8}$ in. and 50 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus 7.00.
B, 50 per cent $\frac{3}{8}$ to $\frac{3}{4}$ in. and 50 per cent $\frac{3}{4}$ to $1\frac{1}{2}$ in.; fineness modulus 7.50.

Number of Shot	Abrasion of Aggregate											
	Change in Fineness Modulus						Wear, per cent by weight finer than No. 14 sieve					
	Gravel	Limestone	Granite	Sandstone	Slag	Average	Gravel	Limestone	Granite	Sandstone	Slag	Average
1800 REVOLUTIONS: GRADING A												
0	0.09	0.17	0.08	0.82[a]	0.19	0.13	1.0	2.0	1.0	14.7[a]	2.5	1.6
2	0.25	0.30	0.19	....	0.25	0.25	3.0	3.6	2.1	....	2.9	2.9
4	0.38	0.43	0.24	1.81[a]	0.40	0.36	4.9	5.6	3.1	35.1[a]	5.0	4.6
6	0.58	0.60	0.38	2.15[a]	0.50	0.51	7.8	7.2	4.2	43.9[a]	6.1	6.3
8	0.77	0.68	0.48	....	0.63	0.64	11.0	9.6	6.3	....	8.1	8.8
10	0.91	0.80	0.60	....	0.76	0.77	13.3	11.2	7.5	....	11.0	10.8
12	1.12	0.97	0.72	....	0.83	0.91	16.4	13.3	10.2	....	10.8	12.7
15	1.30	1.11	0.90	2.86[a]	1.05	1.09	20.0	16.2	12.7	54.3[a]	15.0	16.0
Average	0.67	0.63	0.45	....	0.58	0.58	9.7	8.6	5.9	....	7.7	8.0
1800 REVOLUTIONS: GRADING B												
0	0.05	0.13	0.11	....	0.21	0.12	0.5	1.2	0.7	....	2.6	1.2
2	0.12	0.25	0.18	....	0.26	0.20	0.7	2.1	1.2	....	3.0	1.8
4	0.18	0.38	0.30	....	0.35	0.30	1.3	3.1	1.9	....	3.9	2.6
6	0.19	0.44	0.34	....	0.41	0.34	1.4	3.6	2.4	....	4.7	3.0
8	0.28	0.53	0.40	....	0.44	0.41	2.3	4.8	3.0	....	5.0	3.8
10	0.36	0.63	0.49	....	0.52	0.50	3.3	6.2	4.1	....	5.7	4.8
12	0.37	0.72	0.55	....	0.56	0.55	3.7	7.4	5.0	....	6.3	5.6
15	0.46	0.79	0.62	....	0.60	0.62	4.8	8.7	5.7	....	6.8	6.5
Average	0.25	0.48	0.37	....	0.42	0.38	2.2	4.6	3.0	....	4.8	3.6
10,000 REVOLUTIONS: GRADING B												
0	0.10	0.21	0.18	....	0.41	0.22	0.8	1.3	1.2	....	4.4	1.9
2	0.23	0.57	0.55	....	0.69	0.51	2.2	4.5	5.0	....	7.9	4.9
4	0.52	1.02	0.86	....	1.11	0.88	5.1	10.6	9.1	....	12.7	9.4
6	0.84	1.46	1.03	....	1.42	1.20	10.3	16.9	12.9	....	16.9	14.2
8	1.01	1.90	1.46	....	1.62	1.50	13.0	24.5	18.7	....	20.2	19.1
10	1.43	2.28	1.54	....	1.88	1.80	19.2	30.8	20.2	....	23.7	23.5
12	1.57	2.48	1.91	....	2.12	2.02	21.5	33.5	25.4	....	27.1	26.9
15	2.09	2.66	1.98	....	2.23	2.24	29.4	37.0	26.9	....	29.3	30.6
Average	0.97	1.51	1.19	....	1.44	1.30	12.7	19.9	14.9	....	17.8	16.3

[a] Omitted from average.

TABLE V.—EFFECT OF GRADING OF SAMPLE ON THE ABRASION OF AGGREGATE.

Weight of Sample, 5000 g.
Abrasive charge, six 1⅞-in. cast-iron balls.
Revolutions of cylinder, 1800.

Grading of Aggregate, per cent of each size			Fineness Modulus of Aggregate	Abrasion of Aggregate					
				Change in Fineness Modulus			Wear, per cent by weight finer than No. 14 sieve		
Fine (4 to $\frac{3}{8}$)	Medium ($\frac{3}{8}$ to $\frac{3}{4}$)	Coarse ($\frac{3}{4}$ to $1\frac{1}{2}$)		Gravel	Limestone	Average	Gravel	Limestone	Average
EFFECT OF VARYING PROPORTIONS OF FINE AND COARSE									
0	0	100	8.00	0.15	0.27[a]	0.21	1.2	2.0[a]	1.6
10	0	90	7.80	0.32	0.38	0.35	4.0	4.4	4.2
20	0	80	7.60	0.38	0.52	0.45	5.0	5.6	5.3
30	0	70	7.40	0.51	0.61	0.56	6.9	6.0	6.4
40	0	60	7.20	0.55	0.61	0.58	7.0	6.8	6.9
50	0	50	7.00	0.58	0.62	0.59	7.8	6.8	7.0
60	0	40	6.80	0.59	0.56	0.58	6.7	6.4	6.6
70	0	30	6.60	0.56	0.59	0.58	6.0	6.2	6.1
80	0	20	6.40	0.53	0.50	0.52	4.9	4.9	4.9
90	0	10	6.20	0.48	0.43	0.45	3.8	4.1	4.0
100	0	0	6.00	0.41	0.39	0.40	2.9	3.2	3.0
EFFECT OF VARYING PROPORTIONS OF MEDIUM AND COARSE									
0	0	100	8.00	0.15	0.27[a]	0.21	1.2	2.0	1.6
0	25	75	7.75	....	0.44	0.44	...	3.5	...
0	30	70	7.70	0.19	....	0.19	1.7	...	...
0	50	50	7.50	0.19	0.44	0.28	1.4	3.6	2.2
0	70	30	7.30	....	0.41	0.41	...	3.5	...
0	100	0	7.00	0.18	0.44	0.31	1.2	3.0	2.1
EFFECT OF VARYING PROPORTIONS OF FINE AND MEDIUM									
0	100	0	7.00	0.18	0.44	0.31	1.2	3.0	2.1
30	70	0	6.70	0.30	0.48	0.39	2.2	4.3	3.2
50	50	0	6.50	....	0.49	0.49	...	3.9	...
70	30	0	6.30	0.35	....	0.35	3.2	...	...
75	25	0	6.25	....	0.41	0.41	...	3.8	...
100	0	0	6.00	0.41	0.39	0.45	2.9	3.2	3.0
EFFECT OF VARYING PROPORTIONS OF FINE, MEDIUM AND COARSE									
5	90	5	7.00	....	0.69	0.69	...	3.4	...
10	0	90	7.80	0.32	0.38	0.35	4.0	4.4	4.2
10	10	80	7.70	0.34	0.45	0.40	3.7	4.5	4.1
10	30	60	7.50	0.32	0.43	0.38	3.8	4.6	4.2
10	50	40	7.30	0.33	0.42	0.38	3.1	4.2	3.6
10	80	10	7.00	0.22	0.47	0.34	1.9	3.7	2.8
15	70	15	7.00	0.23	0.48	0.36	2.1	4.1	3.1
20	0	80	7.60	0.38	0.52	0.45	5.0	5.6	5.3
20	20	60	7.40	0.41	0.49	0.45	5.0	6.1	5.6
20	30	50	7.30	0.41	0.56	0.48	4.8	5.5	5.1
20	60	20	7.00	0.27	0.51	0.39	2.6	4.4	3.5
25	25	50	7.25	....	0.57	0.58	...	5.8	...
25	50	25	7.00	0.33	0.54	0.44	2.9	4.8	3.8
30	0	70	7.40	0.51	0.61	0.56	6.9	6.0	6.4
30	10	60	7.30	0.48	0.58	0.53	6.6	7.0	6.8
30	40	30	7.00	0.40	0.59	0.50	4.8	5.6	5.2
30	60	10	6.80	0.33	0.48	0.40	3.0	4.5	3.8
30	70	0	6.70	0.30	0.48	0.39	2.2	4.3	3.2
33.3	33.3	33.3	7.00	....	0.60[b]	0.60	...	5.9[b]	...
35	30	35	7.00	0.45	0.59	0.52	5.6	5.9	5.8

[a] Average of two tests.
[b] Average of five tests.

TABLE V.—EFFECT OF GRADING OF SAMPLE ON THE ABRASION OF AGGREGATE—(*Continued*).

Grading of Aggregate, per cent of each size			Fineness Modulus of Aggregate	Abrasion of Aggregate					
				Change in Fineness Modulus			Wear, per cent by weight finer than No. 14 sieve		
Fine (4 to $\frac{3}{8}$)	Medium ($\frac{3}{8}$ to $\frac{3}{4}$)	Coarse ($\frac{3}{4}$ to $1\frac{1}{2}$)		Gravel	Limestone	Average	Gravel	Limestone	Average
EFFECT OF VARYING PROPORTIONS OF FINE, MEDIUM AND COURSE (*Cantiuued*)									
40	0	60	7.20	0.55	0.61	0.58	7.0	6.8	6.8
40	30	30	6.90	0.43	0.59	0.52	5.0	5.6	5.7
40	20	40	7.00	0.53	0.62[b]	0.58	6.7	6.6	6.6
45	10	45	7.00	0.57	0.61[b]	0.59	7.3	6.1[b]	6.7
50	0	50	7.00	0.58	0.60	0.60	7.8	7.2	7.0
50	15	35	6.85	....	0.57	0.57	...	6.3	...
50	25	25	6.75	....	0.51	0.51	...	5.3	...
50	40	10	6.60	0.40	0.52	0.46	4.6	4.9	4.8
50	50	0	6.50	....	0.49	0.49	...	3.9	...
60	30	10	6.50	0.35	0.53	0.44	3.9	4.9	4.4
70	0	30	6.60	0.56	0.59	0.58	6.0	6.2	6.1
70	10	20	6.30	0.46	0.54	0.50	5.6	5.5	5.6
70	20	10	6.40	0.35	0.48	0.42	3.8	4.9	4.4
70	30	0	6.30	0.35	....	0.35	3.2	...	...

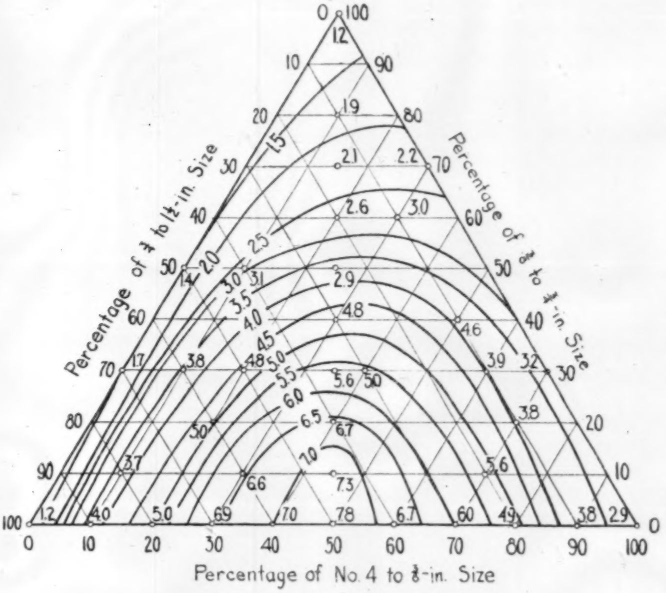
[b] Average of two tests.

Figs. 4 and 5 raises the question as to whether a 5000-g. sample is larger than should be used.

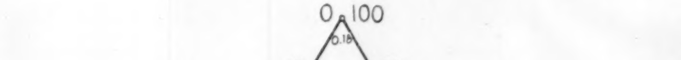
In selecting the size of sample to be used the difference in the action of the test for large and small samples should be considered. In the larger samples the action more nearly approaches simple abrasion, while in the smaller samples the impact of the iron balls and of the larger aggregate particles undoubtedly play an important part. Further studies in which the abrasive charge and duration of test are varied for the different weights of sample would be of interest.

Effect of Number of Balls.—The effect of number of cast-iron balls in the abrasive charge is shown in Table IV and in Figs. 6 and 7. The nature of these curves is very similar to those in Figs. 2 and 3, which show the effect of number of revolutions, indicating that an increase in the abrasive charge had about the same effect on the amount of wear as an increase in the duration of the time of test. As in the case of the variation in the number of revolutions, the effect of grading seemed to be more important in the case of the gravel than of the other materials. In this case also, grading *A* shows a higher percentage of wear for the gravel than for the crushed limestone, while grading *B* shows the limestone to be higher than the gravel.

Effect of Grading of Test Sample.—The results of abrasion tests on fifty-seven different gradings of gravel and of crushed limestone are



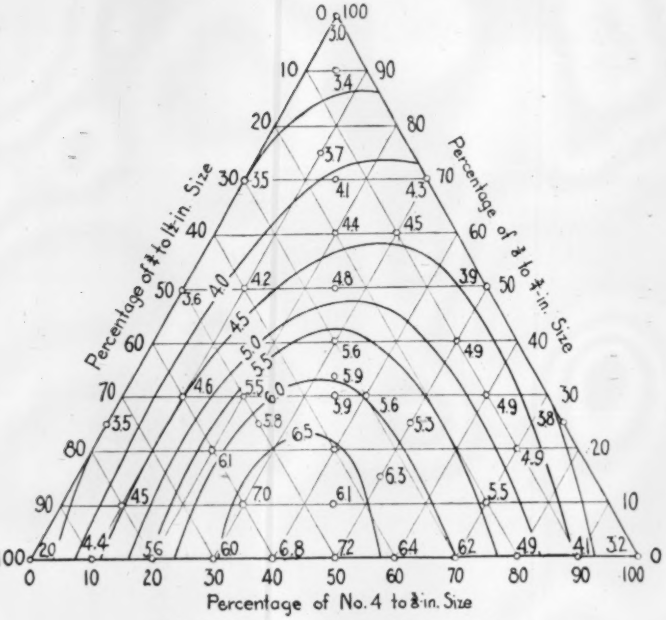
(*a*) Percentage of wear.



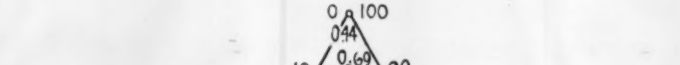
(*b*) Change in fineness modulus.

FIG. 8.—Effect of Grading of Gravel on Amount of Wear.

Data from Table V.



(*a*) Percentage of wear, limestone.



(*b*) Change in fineness modulus, limestone.

FIG. 9.—Effect of Grading of Limestone on Amount of Wear.
Data from Table V.

shown in Table V and Figs. 8 and 9. The different gradings were obtained by varying the proportions of fine, medium and coarse sizes of material (No. 4 to $\frac{3}{8}$ in., $\frac{3}{8}$ in. to $\frac{3}{4}$ in., and $\frac{3}{4}$ in. to $1\frac{1}{2}$ in.). They were tested using a 5000-g. sample run for 1800 revolutions with six $1\frac{7}{8}$-in. cast-iron balls added as an abrasive charge.

TABLE VI.—EFFECT OF SHAPE OF PARTICLES ON WEAR OF AGGREGATE.[a]

Tests made in a standard Deval abrasion machine
Standard sample made up of 50 pieces weighing 5000 g.
Machine run for 10,000 revolutions at about 33 r.p.m.

Kind of Aggregate	Wear, per cent by weight finer than No. 16 sieve							
	First Test	Second Test	Third Test	Fourth Test	Fifth Test	Sixth Test	Seventh Test	Eighth Test
Crushed Limestone (Chicago, Ill.)	3.82	2.86	....	....	....	....	....	....
	3.86	2.66	2.22	1.76	....	....	....	....
	3.90	3.04	2.94	2.20	1.88	1.68	1.36	1.44
	3.88	3.04	2.26	2.00	1.74	1.68	2.04	1.28
Average	3.86 (100)[b]	2.80 (72)	2.44 (63)	1.99 (52)	1.81 (47)	1.68 (43)	1.70 (44)	1.36 (35)
Crushed Slag (Chicago, Ill.)	7.38	....	....	....	....	....	....	....
	7.24	4.00	2.80	....	....	....	....	....
	7.68	3.68	2.74	2.30	2.60	2.04	....	....
	7.50	3.60	2.86	2.50	2.20	2.20	2.00	1.92
Average	7.45 (100)	3.76 (50)	2.80 (38)	2.40 (32)	2.40 (32)	2.12 (28)	2.00 (27)	1.92 (26)
Pebbles[c] (Elgin, Ill.)	1.94	1.74	....	....	....	....	....	....
	1.86	2.00	1.60	1.60	1.64	1.30	....	....
	2.06	1.80	1.70	1.60	1.56	1.40	1.36	....
	2.29	1.74	1.60	1.60	1.08	0.92	1.46	....
Average	2.08 (100)	1.82 (88)	1.63 (78)	1.60 (77)	1.43 (69)	1.21 (58)	1.41 (68)	
Crushed Sandstone (Kettle River, Minn.)	11.50	....	....	....	....	....	....	....
	11.56	7.70	....	....	....	....	....	....
	12.10	5.20	5.60	5.34	....	....	....	....
	11.20	8.66	6.30	4.56	5.54	5.60	....	....
Average	11.59 (100)	7.19 (62)	5.95 (51)	4.95 (43)	5.54 (48)	5.60 (48)		
Crushed Granite (Wisconsin)	1.70	....	....	....	....	....	....	....
	1.78	0.56	0.40	....	....	....	....	....
	1.64	0.80	0.56	0.34	0.32	....	....	....
	1.72	0.60	0.40	0.46	0.32	....	....	....
Average	1.71 (100)	0.65 (38)	0.45 (26)	0.40 (23)	0.32 (19)			

[a] Data from discussion by Duff A. Abrams, *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 370 (1922).
[b] Per cent of wear for first test.
[c] Pebbles of limestone origin.

The general trend of the data can be studied most conveniently from Figs. 8 and 9, where proportions of fine, medium and coarse are shown on a triaxial diagram and the amounts of wear represented by contour lines. Figure 8 shows the tests for the gravel, giving the percentages of wear and differences in fineness modulus respectively. Figure 9 gives the results for crushed limestone in a similar manner.

The grading had an important effect on the amount of wear, as an inspection of the charts will show. The greatest variations due to grading were found for the tests in which the proportions of fine to coarse sizes were changed and the medium size omitted. In the case of the gravel, for example, the percentage of wear increased from about 1.5 to 7.5 as the proportions of fine mixed with the coarse were increased from 0 to 50 per cent. For amounts of fine material greater than 50 per cent, the wear decreased. A similar relation was also found in the case of the crushed limestone.

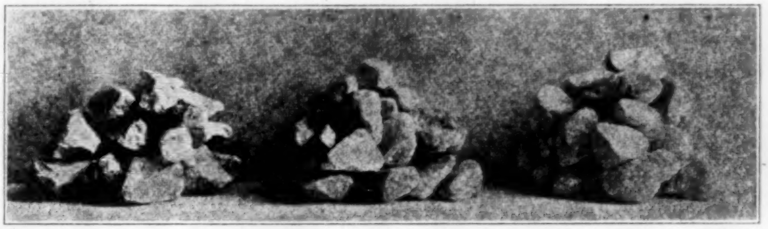
Variations in the proportions of two "adjacent" sizes (medium and coarse, or fine and medium) had very much less effect on the amount of wear than in the case of mixtures of fine and coarse with the intermediate size omitted. In fact, varying the proportions of the two coarser sizes had practically no effect on the wear; also, comparatively small changes were found for different combinations of the two finer sizes.

The contour lines show that a great many different gradings may be used to obtain the same percentage of wear. For example, in the case of gravel, 4½ per cent of wear was obtained for the following gradings, and the infinite number of intermediate gradings falling in the same path:

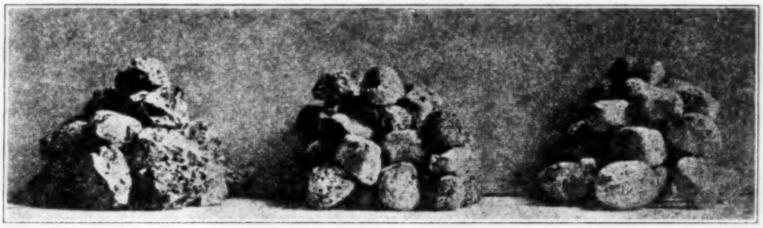
No. 4 to $\frac{3}{8}$ in., per cent	$\frac{3}{8}$ in. to $\frac{3}{4}$ in., per cent	$\frac{3}{4}$ in. to 1 $\frac{1}{2}$ in., per cent
15	0	85
16	14	70
20	35	45
30	44	26
40	42	18
55	30	15
60	25	15
70	15	15
85	0	15

The studies thus far carried out have not shown what characteristic is held in common by these different gradings. Off-hand inspection will show that it is not the fineness modulus, surface area nor number of particles. Experience with other investigations indicate that contours showing density or void content of these aggregates would assume approximately the same shape as these showing abrasion. Further tests, now under way, should throw some light on the question of whether the density of the aggregate plays as important a part in determining the amount of wear of the aggregates in the Deval test as this suggestion would imply. Probably the relationship is so complicated by the varying combinations of impact and abrasion in the test, that it will be difficult to establish.

As has been pointed out in other places in this paper, the effect of grading was more important in the case of the gravel than of the limestone. This is made apparent by the greater frequency of the contour lines in Fig. 8 than in Fig. 9.



(*a*) Before testing. (*b*) After 1 test. (*c*) After 8 tests.
Crushed Limestone.



(*d*) Before testing. (*e*) After 1 test. (*f*) After 8 tests.
Crushed Blast-Furnace Slag.

FIG. 10.—Samples of Aggregates Used in Deval Tests.
Data of tests of aggregate given in Table VI.

Effect of Shape of Particles on Wear of Aggregates.—The discussion of the data in Table VI showing the effect of shape of particle, which follows, is taken verbatim from a brief discussion by Duff A. Abrams which appears in the 1922 *Proceedings* of the Society,[1] except that references to tables and figures have been modified to conform to this paper.

"The purpose of the tests in Table VI was to study the wear of aggregates of different types after the crushed particles had been worn to a rounded form. All tests were made in the Deval abrasion machine in accordance with the Standard Method of Test for Abrasion of Rock

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[1] Duff A. Abrams, Discussion on Test for Gravel, *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 370 (1922).

(Serial Designation: D 2 – 26) of the American Society for Testing Materials. Four standard samples of each aggregate were run at first in order to secure a sufficient quantity for the later tests.

"Figure 10 shows the shape of particles of limestone and crushed slag before testing and after the corners had been rounded off. It should be pointed out that this photograph shows only the large-size particles, similar to those used in the standard sample.

"The average percentage of wear of limestone pebbles for the fifth to seventh tests is almost the same as for the crushed limestone at the eighth test, although the latter value is only 35 per cent of the former in the first test.

"It is significant that the *order* of values for the different aggregates is the same in the fifth as in the first test.

"It is probable that the particles used in the later tests represented the *hardest* and *toughest* of the original samples."

CONCLUSIONS

The following are the principal conclusions which may be drawn from the data:

1. The differences in the percentages passing sieves suitable for measuring the dust of abrasion indicate the desirability of taking into account more than one sieve in measuring the wear.

2. Measurement of the differences of the fineness modulus before and after the test provides a convenient method for taking into account the amounts passing (or retained on) all of the sieves used in making the sieve analysis.

3. For given groups of tests the differences in fineness modulus are approximately proportional to the percentage of wear as measured by the amount passing the No. 14 sieve.

4. The wear of the aggregates increased with the number of revolutions of the cylinder. For the harder materials this increase was approximately proportional to the duration of the test. For the softer materials and the longer testing periods the rate of abrasion decreased with the increase in the number of revolutions, showing the cushioning effect of the dust of abrasion.

5. The amount of wear was decreased by increasing the weight of the sample. The differences in the amount of wear for different aggregates was greater for the small samples than for the larger ones. For the conditions of these tests and for samples greater than about 5000 g., little difference in amount of wear was found for the three different aggregates tested in this group.

6. The tests indicate that for large samples the abrasive action of the testing machine was more prominent, while for the smaller samples the impact action of the abrasive charge and of the larger aggregate particles had an important effect on the amount of wear.

7. The effect of increasing the number of cast-iron balls in the abrasive charge was similar to increasing the number of revolutions of cylinder.

8. For tests of different gradings made up of combinations of fine, medium and coarse sizes (No. 4 to $\frac{3}{8}$ in., $\frac{3}{8}$ in. to $\frac{3}{4}$ in., and $\frac{3}{4}$ in. to $1\frac{1}{2}$ in.) the greatest amount of wear was obtained for a sample consisting of approximately equal parts of fine and coarse with the medium size omitted.

9. Varying the proportions of fine and coarse caused great differences in the amount of wear. In the case of gravel, the wear was increased from about 1.5 to 7.5 per cent by increasing the proportion of fine from 0 to 50 per cent. Addition of the fine material in excess of 50 per cent decreased the wear.

10. Variations in the proportions of medium and coarse, omitting the fine size, or fine and medium, omitting the coarse size, had relatively little effect on the amount of wear.

11. The same amount of wear was obtained for a great many different gradings, but no characteristic common to all of the gradings was developed by these tests.

12. Variations in the amount of wear due to grading were greater in the case of the gravel than of the other aggregates tested.

13. The amount of wear for angular aggregates was greatly decreased by rounding the particles before the test.

DISCUSSION

Mr. Jackson. MR. F. H. JACKSON[1] (*presented in written form*).—Although the standard Deval abrasion test was first developed as a test for quality of stone to be used in macadam road construction, it had become so well established by the time the concrete road was first introduced that it seemed reasonable to use the test for concrete aggregates also. It has consequently found its way into practically all of the state highway department specifications for concrete pavements. The original requirements naturally followed the values which had previously been in use. However, as time went on and the results of practical experience became available it became increasingly evident that much less rigid requirements might be employed with safety. This was, of course, due to the fact that the rapidly decreasing percentage of abrasive traffic on our highways made the matter of wear resistance of less and less importance. Engineers too are coming more and more to the belief that soundness or durability is of more importance than resistance to wear when the stone is to be used as concrete aggregate. It is true, however, that the softest rock are apt to be the least durable and for this reason alone the abrasion test may still be justified as an indirect means for guarding soundness.

With all its faults, this test has persisted with no change for over 40 years, and is now practically the universal method for rating the products of crushed stone quarries. As originally devised, however, it was adaptable only to rock. The necessity of having a means for rating other materials, such as gravel and slag, offered for use in the same types of construction soon led to efforts to modify the test so as to take care of these materials. A modification of the test first proposed by A. S. Rea of Ohio to make it satisfactory for gravel has been in use in several states for a number of years. Quite recent efforts to improve this test by making it adaptable to various sizes of gravel have resulted in the presentation by the Society's Committee D-4 on Road and Paving Materials this year of a Tentative Method of Test for Abrasion of Gravel.[2]

It is obvious that specification limits for gravel subjected to this test should be set so as to provide material of comparable quality to crushed stone, in so far at least as resistance to wear is concerned.

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[1] Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 28, Part I, p. 940 (1928).

Mr. Jackson.

Unfortunately, however, outside of Ohio very little systematic effort has apparently been made to determine the relationship between the two tests. Considerable work was done by Mr. Rea originally in order to determine the proper limits for use in his state. In the matter of abrasion test limits, as in all others involving the selection of local materials, the question of specification limits usually resolves itself into the use of a value which will admit the best locally occurring material—with some rather indeterminate value set as the absolute maximum. On this basis, values for abrasion loss in the gravel test have been set anywhere from 5 to 15 per cent in various parts of the country depending on available material—with 20 per cent as the upper limit.

Without experimental proof, it has been tacitly assumed that, due to the severe impact action of the cast-iron balls composing the abrasive charge, the percentage of wear in the gravel test should be set at from two to three times the value used for crushed stone of comparable quality. With the idea of securing some experimental data along this line, the Bureau of Public Roads ran a series of tests, the results of which may be of some interest. A number of samples of rock varying in percentage of wear from 2 to 20 in the standard Deval test were broken into fragments varying from ½ in. to 3 in. in size and run in a standard brick rattler with an abrasive charge consisting of 200 lb. of ½-in. and 1-in. steel cubes until the edges had been rounded into the semblance of natural gravel. The rounded samples were then run as gravels in the modified abrasion test, using six 1⅞-in. cast-iron balls, and the material graded as called for in the method. The results of these tests are shown in the accompanying Table I. It is quite interesting to note than in practically every case the results of the modified test, using cast-iron balls and rounded fragments, are actually lower than the results of the standard Deval test. These results are surprising to say the least and indicate either that the

TABLE I.—COMPARISON BETWEEN STANDARD AND GRAVEL ABRASION TESTS.

	Standard Test, Percentage of Wear	Gravel Test, Percentage of Wear	Ratio, Gravel Test to Standard Test		Standard Test, Percentage of Wear	Gravel Test, Percentage of Wear	Ratio, Gravel Test to Standard Test
No. 1	7.4 (5)[a]	4.0 (5)[a]	0.54	No. 10	6.3 (5)	...	....
No. 2	4.5 (5)	1.8 (5)	0.40	No. 11	4.4 (5)	1.7 (5)	0.39
No. 3	4.7 (5)	1.6 (5)	0.34	No. 12	7.0 (5)	7.7 (5)	1.10
No. 4	4.9 (5)	2.9 (3)	0.59	No. 13	8.9 (5)	7.3 (5)	0.82
No. 5	6.1 (5)	4.9 (4)	0.80	No. 14	11.7 (2)	5.5 (3)	0.47
No. 6	8.1 (5)	22.6 (5)	2.79	No. 15	2.6 (1)	2.7 (3)	1.04
No. 7	19.7 (5)	13.8 (3)	0.70	No. 16	5.8 (1)	1.4 (3)	0.24
No. 8	4.1 (5)	3.4 (5)	0.83	No. 17	11.0 (3)	13.7 (3)	1.24
No. 9	8.7 (5)	6.0 (6)	0.69	No. 18	4.0 (1)	6.9 (3)	1.72

[a] Figures in parentheses indicate number of individual tests averaged.

Mr. Jackson. impact action of the shot is not so severe as has been supposed, at least in so far as material of uniform quality is concerned, or that gravels in general are not nearly so resistant to this action as crushed stone.

It will be admitted, however, that the synthetic gravels prepared for the test although having rounded edges and corners were much more uniform in quality than the average natural gravel, especially those of glacial origin. In the writer's opinion, this emphasizes a fundamental weakness in the gravel abrasion test which is most apparent when non-uniform materials are being investigated. The results of an abrasion test such as this can only hope to show the *average* loss by abrasion of the entire sample and in no sense indicates the uniformity of the material, which is really the important thing. In other words, a gravel composed of fragments of reasonably uniform composition with a loss of say 10 per cent, is an entirely different proposition from a gravel composed of fragments with a wide variation in hardness, the average wear on which may still be 10 per cent. It has been suggested that the gravel abrasion test be still further modified by running two tests on the material, the second test on the residue from the first, and that the difference between the two will be an indication of the uniformity of the material. This is, of course, on the assumption that very soft or friable pieces will be eliminated in the first test. Another possibility is to use the concrete strength test as a method of rating coarse aggregates for concrete, together with either a static load or an impact test to be applied to individual pieces with the idea of obtaining the percentage of soft and disintegrated fragments in the material—which is after all the vital thing. The static load test for soft pieces has been used successfully in Iowa, whereas the impact test has been used to a considerable extent by the Bureau of Public Roads. Both methods are described in detail in U. S. Department of Agriculture *Bulletin No. 1216*, Revised, the official methods of test of the American Association of State Highway Officials.

In conclusion, the writer believes that the fact that many concrete pavements which are known to have been constructed with gravels showing very high abrasion losses are still in excellent shape after many years' service is an added indication that the modified abrasion test for gravel can not be applied under any and all conditions without raising the limits so high as to be absurd even though it may have proved entirely satisfactory under certain conditions, as notably in Ohio.

METHODS OF PARTICLE SIZE DETERMINATION

By Lincoln T. Work[1]

Synopsis

The aims in fineness measurement of ground minerals, pigments, fuels, and cements have been noted. Methods of determining the desired size function of these materials are classed as direct measurement and indirect comparison. Direct measurement comprises: (1) sieving, (2) sedimentation or elutriation, (3) microscopic sizing and, (4) combinations of these. Indirect comparison comprises: (1) settling rate in a fluid medium, (2) apparent density or packing volume, (3) hiding or light dispersing power, (4) rate of solution, (5) X-ray dispersion, (6) plasticity and (7) effects in products, such as, rubber, putty, etc. The methods are discussed with respect to principles of operation, technique, data and methods for its representation, and limits of application. Direct methods are time consuming, but should be regarded as the fundamental basis of fineness measurement. For practical purposes, a suitable and rapid indirect method may be used, preferably after checking it by direct measurement.

Introduction

Interest in determining the fineness of crushed or precipitated aggregates is of wide extent and great diversity. Particular emphasis has been laid recently on methods for testing powders which are primarily of less than sieve mesh. Even in the coarser sizes, however, greater attention is being paid to fineness than formerly. The main fields in which fineness measurement is important, as well as the reasons for its importance, are here summarized.

1. Grinding ore and stones;
2. Paints and lacquer pigments;
3. Solid Fuels;
4. Cements.

Some special characteristic of fineness is likely to be stressed in each of these fields. When ores are ground for flotation they must be crushed sufficiently to release the individual particles. If they are ground finer, there will be losses. Consequently uniformity of size is the important factor in the preparation of material for flotation. The requirement is different when minerals are ground for leaching or extraction of valuable ore. In that case, solvent action is rapid for the very finest particles, but the recovery is low unless the largest particles are completely leached. These large particles require violent agitation to keep them in contact with fresh liquid and time to complete the extraction. Under such circumstances, grinding is undertaken primarily for the elimination of coarse particles. This same requirement holds in the case of ground ores which are to be used as mineral fillers in rubber, asphalt and paint products. In addition, a certain amount of fine material is required for these fillers. According to one view, the particles should be proportioned in the various sizes so as to give a desired packing volume or apparent density; while, according to other views, total surface or a value of average diameter controls their application. The requirements for pigments are quite diverse. The most significant factors seem to be a minimum of coarse material for smooth texture of the paint and a definite amount of surface for its relation both to wetting by the vehicle and to the hiding or covering power of the paint. Solid fuels are sized for two purposes: in the lump form to determine the suitability for use on various types of grates and feeding devices, and in the pulverized form to estimate the combustibility of the dust when mixed with air. The trend with pulverized coal is to demand extreme fines which carry the flame as well as to limit the maximum size of particle. In cement, the hydraulically active material is of interest, and it may be measured in terms of total surface or of fine material. Actually, the amount of coarse cement is measured, and the inference is made that the fineness is greater with the lesser amounts on the specified sieve.

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[1] Instructor in Chemical Engineering, Columbia University, New York City.

These general requirements determine what is needed to define the fineness of a given material. Thus, the fineness may be expressed by the distribution curve or by a constant. Distribution curves present the amounts of particles of each of the several sizes, in terms of weight, volume, or frequency. These curves have been used as a basis for calculating average diameter, total surface, and other factors. Fineness constants give values representing single properties of the material, such as average diameter, total surface, fineness modulus, apparent density, plasticity, hiding or covering power, and others more directly related to its application. Such measurements are frequently made with but little difficulty and with a single observation, at a considerable saving of time over the determination of the distribution curve. In many cases, the function measured is more directly useful than the fineness distribution curve.

It is the purpose of this paper to study critically the typical methods of fineness determination used for ground ores, paint pigments, mineral fillers, solid fuels, and cements, to secure the actual distribution curve or a fineness constant.

METHODS OF MEASUREMENT OF PARTICLE SIZE

Methods for the determination of fineness may be classed under two main heads: direct measurement and indirect comparison. As these methods are discussed, it will become clear that the difference lies in the fact that the former uses experimental observation with relatively minor assumptions while the latter depends upon the exactness of general laws for the behavior of fine particles or of data not directly interpretable by scientific laws. The line is drawn between these two methods at the point where the uncertain factors begin to play some part in the ultimate accuracy of the work. In general, the direct method measures the distribution curve; while the indirect method gives a fineness constant.

For the direct method, three typical operations are outstanding in their application:

1. Sieving;
2. Elutriation or sedimentation;
3. Microscopic measurement.

For the indirect comparisons, the following characteristics typify the bases used:

1. Settling rate in a fluid medium;
2. Apparent density or packing volume;
3. Hiding or light dispersing power;
4. Rate of Solution;
5. X-ray dispersion;
6. Plasticity;
7. Effects in products, such as rubber, putty, etc.

DIRECT MEASUREMENT OF DISTRIBUTION CURVE

Any of the methods already enumerated, that is, sieving, elutriation, or microscopic count, may be used to determine fineness distribution. Each of them is discussed here with respect to principles of operation, technique, and special application. As these methods have special applications to certain ranges of size, it is sometimes more convenient to express coarse material in terms of sieve data and fine material in terms of elutriation or of microscopic count. The means of making these values comparable with each other are presented herein as combination methods.

Sieving:

As an analytical test, sieving measures the fineness of a mixture by separating it into particles of different sizes. The respective weights may be determined for each of the portions just passing one size of sieve opening and failing to pass the next smaller sieve opening. Such a group of particles may be said to have diameters ranging between the two values of sieve aperture. These weights and diameters are the basis of the distribution curve. In other cases, sieving is used to determine the amount of material coarser than a specified size.

The test has certain empirical features which affect the experimental results, such as the specific particle dimensions, uniformity in manufacture and maintenance of sieve cloth, variation in sieving procedure, and nature of the material with respect to softness and agglomeration.

If a particle presents itself to a sieve opening so that its cross-section is less in all dimensions than the openings it will pass through the opening. Sieve apertures are essentially square in form. Spherical particles which just pass a given sieve must have a diameter which is less than the side of the opening. On the other hand, very flat particles may pass the sieve if the least dimension across the face of the particles is less than the diagonal of the sieve opening. It is apparent that the critical dimension is that of the minimum cross-section.

It is not always a simple matter to present the smallest particle dimensions to one of the sieve openings, for instance flat particles slide across the sieve with but little tendency to stand upright. In some unpublished tests with pieces of wire cut to various lengths, A. M. Gaudin found a critical length for a given sieving procedure above which the minimum cross-section was never exposed to the sieve opening. Thus, the third dimension may play a part in sieving.

The accuracy and reproducibility of the test is affected by the nature and condition of the material itself. The tendency to agglomerate or form clusters often prevents satisfactory separation of the individual particles into their respective sieve sizes. This action may be due to moisture, electrical charges, or physical surface adhesion. This same tendency causes the adherence of dust to sieves as well as to coarse particles. Then separation is poor and the sieves tend to clog. Agglomerates are often broken and dust washed through by sieving the material in water instead of dry. In such a case, the conditions of separation are somewhat different, particularly with respect to presentation of particles to sieve apertures and with respect to fluid films between particles and sieves.

Soft and easily friable materials are not amenable to exact sizing with sieves because of mechanical abrasion which changes the fineness during the test.

The openings in a sieve are not all of exactly the same size. Consequently, the separation will not be perfect until the smallest particle on the sieve cannot pass the largest sieve opening. Such a condition is not actually reached, but it makes time and sieving procedure important variables in the test.

An arbitrary time may be selected such that the change in percentage with normal variation in the test is insignificant. Or, it may be specified that the amount passing the sieve in a given interval of time be a minimum. It must be conceded that neither of these criteria corrects for faulty methods of presenting the particles to the sieve openings, nor for reduction in particle size due to abrasion.

Irregular apertures play a greater part when the sieves are used in nests. A normal variation of 2 microns in 200-mesh cloth is relatively small in its influence when the material ranges up to 70 or 100-mesh. If, however, the 170-mesh sieve, having an aperture of 88 microns, be placed above the 200-mesh sieve, having an aperture of 74 microns, the variation is 15 per cent of the interval and may affect the individual values to that extent. Evidently the limit to the finest test sieves and to closeness of nest intervals lies in the uniformity of sieve cloth.

To obviate the difficulties cited here, sieving equipment and the procedures in the more important fields have been standardized.

Specifications are rigid as to uniformity of wire and weave, particularly in the cases of the finest mesh. This, as well as uniform maintenance and sieving technique, is further controlled by the use of standard samples. The practical limit for analytical testing seems to lie at 200 mesh, although wire cloth is made to 325 mesh and bolting cloth is made for still finer sizing. With certain materials and with good technique, sieves finer than 200 mesh give useful data.

Several schemes for the nesting of sieves have been proposed, most of which decrease the sieve interval with particle size. There are several series in which consecutive sieve openings vary by a constant factor. For the most exact work down to 200 mesh, one in which that factor is 1.189 (or the fourth root of two) may be used. It will be observed that the closeness of this nesting in the finer sizes introduces appreciable errors in the individual values which may usually be eliminated in the distribution curve by visual or statistical smoothing methods.

Sieving procedures may also be classified by fluid, as dry, wet, or wet and dry, or by methods of agitation, as hand or mechanical shaking. The dry process is used for coarse materials and for fines having hydraulic properties. The samples should be oven dried. If dusty, the dust may be brushed from the sieve walls once during the test. This procedure gives abrasion losses and does not take care of persistent agglomeration. Wet sieving overcomes these difficulties but does not give the same close sizing. A combination of wet sieving on the finest screen and dry sieving of the oversize in the nest gives very satisfactory results with all save very soft or hydraulic materials. Hand agitation has been accepted as standard, although there are now several very satisfactory mechanical devices which save labor and eliminate the personal factor.

As an analytical method, sieve sizing is limited to particles essentially larger than 0.0029 in. or 0.074 mm. (74 microns). It is used for ground mineral or stone and also for control of the amount of coarse particles in pulverized materials. It leaves unsized the vast field of fine powders.

Elutriation:

Methods based upon settling velocities in a fluid medium have long been used to separate particles into groups of different sizes. Those which effect separation while the particle falls are known as sedimentation methods, while those which separate by an upward stream of fluid are known as elutriation methods. The fluids, the equipment, and the technique vary; but the weights and diameters of the portions so produced become the basis of the fineness distribution curve.

Two principal relations have been established as controlling the settling velocity of an isolated particle; namely Stokes' law, with a modification for electrical charges on particles of minute size, and the settling relation for turbulent flow of the fluid.

Stokes' law(**1**)[1] shows the constant velocity of fall for a spherical particle in a fluid. It is obtained by equating the general relations for the force of friction and the force of gravity.

$$v = \frac{2}{9} \frac{(P_p - P_l)g}{n} r^2$$

where v = velocity of fall,
P_p = density of particle,
P_l = density of liquid,
n = viscosity of liquid,
g = gravity constant, and
r = radius of particle.

For a given material in a given fluid,

$$v = K\,d^2$$

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[1] The boldface numbers in parentheses refer to the papers given in the list of references appended hereto.

When with the other factors, the constant K includes the effect of particle shape, the relation is useful over most of the range of sub-sieve sizes. Richards gives as the upper limit of particle diameter for this relation 0.20 mm. for quartz in water and 0.13 mm. for galena in water(2). Above this the viscous resistance of the fluid ceases to control and eddy resistance becomes dominant. Stokes' law becomes unreliable in the colloidal range of size, when electrical charges on the particles affect their rate of fall. This effect has been determined quantitatively by Millikan(3).

As the figures given for quartz in water and galena in water indicate, it is only for relatively large particles that viscous flow is supplemented by turbulent flow. The relation then is as follows:

$$v = \frac{K'}{d}$$

Martin has proposed an intermediate relation of $v = K'' d$ for the elutriation of quartz particles in air(4).

Stokes' law is the one which prevails over the range of practical application of sedimentation and elutriation procedures.

Factors, as particle shape, variation of fluid viscosity with temperature, completeness of deflocculation, and hindered settling effects, make the method somewhat uncertain for rigid separation into diametral ranges.

The assumption of Stokes' law that the particles are spherical is in part obviated by the use of an empirical constant. In other words, irregular particles have effective diameters in settling which differ from the spherical or any other microscopically measurable. The constant is then applicable, if the departure from spherical particles at one diameter is statistically the same as the departure from spherical at another. This has not been established in sub-sieve sizes; and, what is more, the present work shows differences in particle form at different sieve sizes.

The viscosity of the fluid enters into Stokes' law and cannot be ignored. While it is recognized that a petroleum oil may be say five times more viscous at 60° F. than at 100° F., it is not generally appreciated that water varies from 0.0066 c. g. s. units at 100° F. to 0.0110 c. g. s. units at 60° F., an amount not quite double the lesser value. Elutriation equipment is seldom controlled as to temperature; and, though a single run may be quite constant, the test results are not uniform with seasonal changes if the effect of viscosity is not taken into account.

The assumptions of settling velocity are based upon individual and homogeneous particles. A cluster of particles containing air inclusions or even some of the fluid in the interstices will behave neither as one large particle nor as the small units of which it is composed. The preparatory treatment of the mixture requires thorough deflocculation.

Though the particles be thoroughly dispersed, if they are not sufficiently far apart, they will not settle independently of each other. This effect is known as hindered settling and shows itself in concentrated slurries by the entire mass settling together and leaving relatively clear supernatant liquid rather than a gradual brightening of the fluid. It is particularly important when the particles are very fine. Elutriation apparatus should have sufficient volume, or the new solid be not exposed too rapidly, else coarse particles will not settle freely through the fine and separation will be inexact.

The procedures may be classified into three primary types: sedimentation, liquid elutriation, and gas elutriation.

A rapid method which gives some results consists in dispersing a solid in a given liquid, allowing it to stand for a calculated length of time, and carefully decanting the fluid to a given level. All particles with a settling velocity sufficiently great to go below that level will remain in the original container. Progressive treatment of the sediment gradually eliminates those particles which cannot settle from the upper level to the lower level in the allotted time. Hindered settling may cause trouble in the test, but this may be obviated by using little solid and much fluid and by allowing a longer time for settling before the first decantation. Such a test is performed in a deep beaker; and it gives a rough separation which should be further sized with the microscope.

Elutriation equipment is used to remove fine particles by an upward stream of fluid. Those particles which travel with the stream are thus separated from those that can settle against it. The apparatus varies with the nature of the fluid. Of the gases, air is most commonly used, while, of the liquids, water, petroleum, and alcohol are employed.

Air elutriators are usually constructed of a vertical metal tube with a cone at the base. The dry weighed solid is placed in the cone and the finer particles removed by a controlled air stream from a jet. The velocity of the air is maintained low and constant for a period and the residue is weighed. The dust is not easily recovered for weighing, as it adheres to filters and even to the tube, but its maximum diameter is determined from a test slide. The air velocity is then increased for a period and the diameter and weight determined(5). Air velocities are high, and abrasion losses are great, especially with soft materials.

Liquid elutriation apparatus is of many forms and varies from the cone having the critical velocity at the top, to the cylindrical form, having a classifying zone. (See Fig. 1.) One stage may be used at different velocities and the effluent recovered, or several stages of different sizes in series. In the latter case, after a continuous stream of fluid has passed through the several stages, of different critical velocities, the flow is stopped and each fraction and the effluent recovered. This has been done by settling and evaporation; but may be done by filtering through paper and igniting the paper or by filtering and washing through alundum crucibles. Exact recoveries may be had for further sizing.

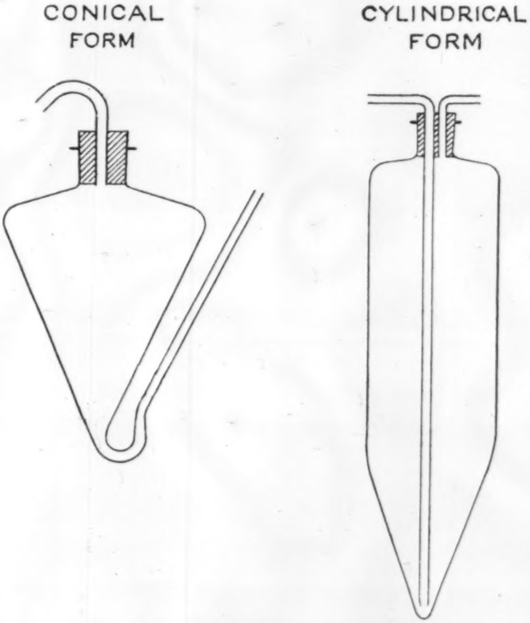

FIG. 1.—Liquid Elutriation Apparatus.

Among its wide applications, elutriation has been used with air for cement and for coal dust, and with fluids for paint pigments, mineral fillers, sands and cements. The extreme limits of size over which it may be used are roughly 5 to 200 microns. The many factors which determine its accuracy make elutriation advisable for approximate classification only. It is now usually supplemented by microscopic control.

Microscopic Measurement:

The use of the microscope to determine the fineness distribution curve has been developed within recent years for paint pigments, photographic emulsions, mineral fillers, cement, and pulverized coal. In brief, the method consists in distributing the powder uniformly over a microscope slide magnifying the particles on different portions of the slide, and counting the numbers of the particles in each of the different sizes. These frequencies, or other calculated relations, such as weight or surface, are the basis of the fineness distribution curve.

The method depends for its accuracy upon the application of good microscopic technology and of accurate statistical methods. Although the working out of the details is controlled by the exact procedure employed, there are a few broad principles which are generally applicable.

Since it is the light image of the particle which is measured in microscopic sizing, the results are exact if the image is properly in focus, clearly defined, and of known magnification.

The definition of the particle is a function of the resolving power of the microscope, as well as of the intensity of illumination and of the shape and opacity of the particle. Abbe proposed that the resolving power, R, varied directly with the numerical aperture of the objective, $N.\,A.$, and inversely as the wave length of light, λ.

$$R = \frac{2\,(N.\,A.)}{\lambda}$$

In turn, the numerical aperture is the product of the index of refraction and the sine of the angle between the light and the optical axis. Besides this, the magnification and consequently the minimum observable size may be limited, the particles may be irregular and therefore subject to bi-refringence, and they may be transparent and approach the refractive index of the medium and thus lose in definition.

Ultimately the resolution is limited by the wave length of light itself. With white light, the limit is approximately one-quarter micron. The ultra-violet light, with its appreciably lesser wave lengths, must be used for the definition of smaller particles.

Penetrating power, or the power to focus through a depth of slide, varies inversely with resolving power. When resolution is high difficulties are encountered in bringing into view particles at different elevations on the slide. This is often avoided by preparing the slide so as to have the greatest number of particles in the same plane.

Magnification is obtained by the lens combination of the microscope and by the projection of the image at a distance. With respect to the lens combination, equipment is made to resolve to the limit of white light for the usual sizes and to use ultra-violet light for the finest sizes of particles. With respect to projection of the image, light intensity and particle definition are the controlling factors.

Statistical treatment is necessary in several places to obviate personal errors or errors from conditions which are not representative. These are briefly:

1. Sufficient material should be taken from a well-mixed sample to be representative of the whole.
2. The slide should be so prepared that all particles are separate so that they may be measured and that large and small particles are uniformly distributed over the area.
3. Check determinations should be made on separate slides.
4. Fields should be selected at random over the entire slide, and they should be sufficiently numerous to be representative.
5. The number of particles measured should be sufficiently large that one or two reported or missing from the fields counted will introduce no serious error in an individual value.
6. At the upper limit of size, the frequency may reasonably drop, to one, two, or three particles; but the number of that size must represent an insignificant portion of the entire weight. A test is likely to be deceptive as to the upper limit of size unless that point is stressed.
7. The intervals of size should be sufficiently small, so that the largest and the smallest particles of any size range may safely be considered alike in size.
8. The intervals may be increased in size with the large particles to obviate an abnormally large count.
9. A simple diametral measurement may be accepted as statistically representative of particle size and for simplicity in the count.
10. Such errors as do creep into individual values should be eliminated by statistical smoothing procedures.

When resolved into their component parts, the different microscopic sizing methods are found to be combinations of a few different procedures at each stage. These steps are:

1. Preparation of the slide;
2. Magnification of the image;
3. Measurement of the particles;
4. Summary of individual frequency measurements.

The preparation of the slide is frequently most exacting in its requirements, and the procedure differs with the size and nature of the material.

Coarse powders may be distributed over a slide by shaking a quantity through a sieve which will pass all particles. If the material is not uniform in size, there may be an uneven distribution of the various sized particles. If it contains dust, there will be loss.

Fine powders, and in general all materials of less than sieve sizes are usually dispersed in some fluid medium. Green(6) describes a method for dispersing paint pigments with redistilled turpentine. When the particles are thoroughly dispersed and uniformly distributed over the slide, the fluid is evaporated. The particles adhere loosely to the glass; and being in the same plane, all of similar size may be brought into focus at once. Other fluids such as diacetone alcohol or some of the lacquer solvents have been successfully used with the pigments. Slides so prepared may be used dry or they may be rendered more permanent and prepared for the oil immersion objective by covering with glycerine and cementing on a cover glass. When the particles have been treated with a thinner like turpentine, their positions are not disturbed by the glycerine. It has even been found possible to use direct immersion in oil with the dry slide, providing that the particles are not rubbed into the immersion oil and that the test is performed quickly.

It seems likely that these thinners leave adsorbed upon the particles a thin film of involatile material. When relatively coarse powders, that is, of little surface, are dispersed with turpentine, the resulting film is sufficiently deep to obscure the particle form. The same turpentine will disperse fine pigments without injury to their form and with some intensification of the image. This may also explain the adhesion of particles to the slide and the failure of glycerine to wet the particles.

Other materials present special problems in dispersion. Most silicate mineral powders can be distributed in water containing a trace of ammonia. Pulverized coal has been dispersed with Zapon lacquer thinner. Pitch dust, which is similar in many respects to coal, is soluble in this solvent. It may be dispersed with a mixture of alcohol and water to which a trace of hydrochloric acid has been added.

Gums, varnishes, and lacquers have also been used for dispersing the powders. In that case, the particles are distributed through the mounting medium. To make focal adjustment simple, the thickness of the film should be less than the penetrating power of the microscope. Otherwise, a counting procedure in which variation of focus is possible during test must be used. The lack of definition observed with some minerals in these gum films is often due to the slight difference in refractive index between the particles and the gum. It may also be due to shrinkage in drying the gum film. The resulting surface is uneven and diffracts the light which would define the particle. A slide so prepared may be improved by coating the first dried film with a second layer of gum or varnish with or even without an added cover glass.

Uniformity of distribution of the different sizes of particles is fully as important as dispersion. This is particularly difficult to obtain with mixtures containing extremely fine and extremely coarse particles. By the method of Green, the coarse particles would move with the rod. If the rod be used vertically, the coarse particles are prone to follow the rod. Large quantities of dispersing fluid are needed and the free liquid draws the particles into clusters about the large ones as it evaporates. Mounting in varnish or gum is preferable to the dry slide when the large particles are of greatest importance. Wise selection of dispersing fluid with respect to its wetting power and its rate of evaporation and careful technique in spreading the particles are another solution to this problem.

Magnification of the particles is obtained by direct observation through the microscope, projection of the microscopic image upon a screen, or by making photomicrographs which are later printed or projected. Weigel(7), Martin(8), and others have used the direct sizing through the microscope. In that method the observer counts the particles of each size, the size being estimated from the squares of an ocular insertion or from a filar micrometer. These devices may be calibrated by means of a stage micrometer. Illumination and definition are usually good, as the magnification is not as great as with the other methods. The method is fatiguing to the eyes and provides no simple means of preventing duplication of some particles in the test.

To increase the magnification and to simplify the measuring procedure, projection methods have been developed. A mirror or prism is attached to the top of the microscope and the image projected onto a ground glass, lacquered glass, or painted wood screen. The microscopic equipment should include apochromatic objectives to give a flat field and condensers and oculars designed to furnish maximum light intensity. The image of the particle is measured by means of a scale prepared by projecting a stage micrometer upon the screen and recording its unit dimension. This may be subdivided to an unlimited extent for the finest particles and expanded to cover the largest particles. The method is simple and makes possible the measurement of particles in a large number of fields.

Green(6), working in the range of the finest pigments, photographs many fields, and projects the slides made from these plates through a stereopticon lantern. Sheppard and Lambert(9), working with photographic emulsions, photograph the fields at a high magnification and then count the particles from the print or enlargement. Either scheme is adaptable to ultra-violet light and is therefore capable of resolution of finer particles than can be observed with white light. It gives a permanent record. At high magnifications, where slight focal adjustment may be necessary for particles in the different layers, some error may be introduced.

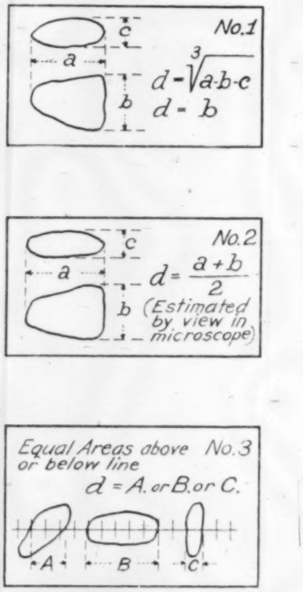
No.1
c
a
b
$d = \sqrt[3]{a \cdot b \cdot c}$
$d = b$
No.2
c
a
b
$d = \frac{a+b}{2}$
(Estimated by view in microscope)
Equal Areas above or below line
No.3
d = A. or B. or C.
A
B
C

FIG. 2.—Critical Dimensions in Microscopic Sizing.

Methods of measurement differ in the diametral criteria, the selection of class intervals, correlating of microscopic magnifications, and simplification of count.

In microscope sizing, the critical dimension varies with the observer. The particle may be said to have three dimensions, length, *a*, breadth, *b*, and thickness, *c*. It usually orients itself on the slide with the *a* and *b* values visible in the microscope. Measurement of these two is simple, whereas measurement of the *c* dimension is more difficult and less accurate. It may be said in passing that this dimension is best measured by a dynamic method in which the particle moves freely in a fluid. Pearson and Sligh(5) have performed some exacting measurements in the vertical on relatively coarse particles.

The simplest of the three typical methods (see Fig. 2) assumes that the small diameter visible in the microscope, that is, the *b* value, is the criterion of measurement. Weigel(7) used the second, in which the diameter was called the average of the two visible in the microscope. Martin(8) aimed at the statistical diameter by assigning to the particle a size equal to the intercept on the filar micrometer, when the scale line divided the particle into equal areas. Inasmuch as there is the same tendency for a particle to orient itself along this line, across it or in any intermediate position, this measures the statistical diameter of the area visible in the microscope, that is, the combination of *a* and *b* dimensions. It does not take account of the tendency for the *c* dimension to lie in a vertical direction.

In the test, all particles must be assigned to their appropriate range of size, x to $x + \Delta x$, known statistically as the class interval. The actual values upon which the interval is based are sometimes determined by the microscope magnification. For instance, a common ocular insert calibrates easily at 17 and 85-micron intervals; and the literature contains data based on ranges which are multiples or fractions of these values. Green has used a millimeter scale which is a simple and easily available instrument but one which may give decimal values of diameter when calibrated with the stage micrometer. In other cases the scale is specially prepared to give integral values.

The size of the class intervals is based upon two schemes—the equal interval and the interval which diminishes regularly or irregularly with diameter. Where the particles do not differ greatly from maximum to minimum diameter as with uniform sizes or with extremely fine sizes the even interval is satisfactory. The counting is not laborious and the frequency curve reaches negligible values in the large size before the total count becomes abnormally high. The reverse is true for mixtures of particles which vary greatly in size. Accuracy demands narrow class intervals in the fine ranges and wider ones in the coarse ranges. In the finer region, the frequency varies from x to $x + \Delta x$ so rapidly that the representative average of the count does not correspond to a diameter of $x + \frac{\Delta x}{2}$. Moreover, particles of size x have mass and volume proportional to x^3 while those of $x + \Delta \tilde{x}$ have mass and volume proportional to $(x + \Delta x)^3$. If Δx is a relatively large proportion of x, the difference in the cubes is appreciable. Hence, it is not accurate to assume that a large frequency of size x, a much smaller frequency of size $x + \frac{\Delta x}{2}$, and a still smaller frequency of size $x + \Delta x$ may be represented as having an average diameter of $x + \frac{\Delta x}{2}$ and an average mass or volume proportional to $\left(x + \frac{\Delta x}{2}\right)^3$. A numerical example where $x = 1$ and $x + \Delta x = 2$ will illustrate this point. (See Fig. 3.)

d	d^3	FREQUENCY	RELATIVE VOLUME OR MASS
$x = 1$	1	1000	1000
$x + \Delta x = 2$	8	250	2000
		1250	3000
$x + \frac{\Delta x}{2} = 1.5$	3.37	1250	4220

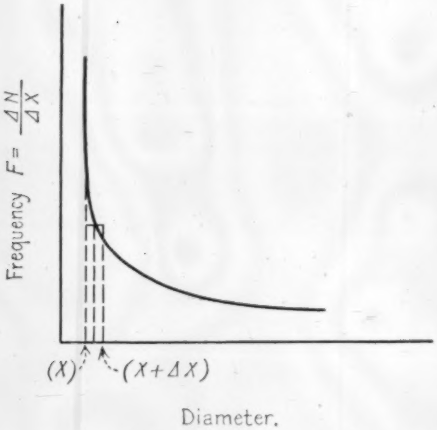

FIG. 3.—Curve Showing Relation Between Diameter and Frequency.

In the coarser sizes, so few particles are to be sized and the difference between $x + \Delta x$ and x becomes proportionately so small that larger intervals are needed. Frequently the coarse particles are counted at a lower magnification, thus requiring a larger actual interval. Thus the same reasoning holds as did in the case of sieve nests in which the interval is varied with diameter. One serious error appears when varying class intervals are taken. It is not the frequency at the average diameter that is recorded, but the sum of the frequencies of each unit diameter in the interval. Increasing the size of the interval increases the total count in the interval. Many eccentric curves appear in the literature through failure to observe this fact, and some conclusions as to inflection in the curve are to be noted which are simply attributable to a change in the size of the class interval. More will be said on this point under Methods of Representation.

TABLE I.—CORRECTED FREQUENCIES.

Calculations for Sample 301-O

Diameter Range Number	Basic Count, 20 fields, 2 slides	Corrective Count, High Power, 12 fields, 2 slides	Corrective Count, Low Power, 20 fields, 2 slides	Corrected Count
12	..	..	1	0.084
11	..	..	4	0.33
10	1	..	12	1.01
9	2	..	42	3.51
8	8	..	107	8.95
7	35	..	384	32.12
	46	..	550	factor (1) $\frac{46}{550} = 0.0836$
6	75	..	..	75
5	165	..	..	165
4	254	..	..	254
3	538	..	..	538
2+	3244	63	177	1 155
2−		114		2 090
1+	..	780	..	14 300
1−	..	1752	..	32 100
				factor (2) $\frac{3244}{177} = 18.33$

TABLE II.—CALCULATIONS OF PERCENTAGE PER MICRON.

Sample 301-O

Diameter Range Number	Corrected Frequency Count (f)	Average Diameter, microns ($d^3 \times 10^{-3}$)	$f \times d^3$	Percentage by Weight	Percentage per Micron
12	0.084	670	56.3	6.48	0.350
11	0.33	244	80.5	9.26	0.501
10	1.01	83.7	84.5	9.72	1.050
9	3.51	30.53	107.0	12.31	1.333
8	8.95	10.53	94.5	10.88	2.354
7	32.12	3.82	122.5	14.09	3.04
6	75	1.31	98.3	11.31	4.89
5	165	0.477	78.6	9.04	3.91
4	254	0.164	41.9	4.82	4.17
3	538	0.0600	32.2	3.59	3.11
2+	1 155	0.0204	23.6	2.72	4.71
2−	2 090	0.00742	15.5	1.78	3.08
1+	14 300	0.00160	22.8	2.62	4.53
1−	32 100	0.000059	1.9	0.22	0.38
			860.1		

NOTE.—Factor for converting individual fd^3 to per cent = 98.96 (per cent less than 200 mesh) ÷ 860.1 (total fd^3 = 0.1150.

The use of different microscopic magnifications is frequently necessitated for mixtures which differ greatly in size. It is a problem to determine the best way to bring these values together into a single test. If a particle of 1 micron is magnified so that the image is 2 mm. in size, a particle of 100 microns becomes 20 cm. At these high magnifications it is rare to find the particles of this size and almost impossible to determine their dimensions exactly. A larger and more representative count may be made on a slide containing more particles and having a lower microscopic magnification. In the finer sizes, it may be desirable to subdivide the usual class interval without the large count which would normally be required at these sizes.

Two methods for doing this are available. Weigel(7) has described in some detail how the areas of the fields visible at each magnification are determined and how the counts of coarse and fine particles are combined on the basis of the counts per field and the ratios of the field areas. In the size intervals where the count overlaps, there are some discrepancies which may be either in the calibration of field areas or, what is more likely, in the variation between fields counted.

The author has employed a method of correlation based on the ground that the total count is more exact than the individual values of which it is comprised. The primary count is made at an intermediate magnification and includes all particles which are visible. A lower magnification is then used for counting a large number of coarse particles. The lower limit of size in this count is determined as the size above which the primary count gives a sufficiently large total count, say 25 to 100 particles, depending upon the accuracy required. The value of the total frequency may be redistributed into individual fractional values of far greater accuracy. Table I shows the method of making the correction in both fine and coarse sizes and Table II the calculation of weights from the data.

Factor 1 for correction of the basic count in the ranges number 7 to 12, inclusive, is based upon a ratio of the respective total frequencies of the basic count and of the low power corrective count. The individual frequency values of the low power corrective count are multiplied by this factor to give the corrected count.

Factor 2 for correction of the basic count in the ranges of 1 and 2 is based upon a ratio of the frequencies in range 2 of the basic count and in ranges 2− and 2+ of the corrective count. The individual frequencies of the corrective count are multiplied by this factor to give the corrected values in ranges 1−, 1+, 2−, and 2+. It has been noticed that the total of 1+ and 1− is greater than that of range 1 of the basic count, showing that at the higher magnification more particles become perceptible to the eye. That is the reason for basing the correction on the frequency in range 2. The individual frequencies in ranges 3 to 6 are sufficiently high to need no correction.

A question may be raised as to the feasibility of counting 300 or 20,000 particles of any one size. This may be obviated with but little uncertainty in the screen projection method. The screen is ruled into sections of equal area. If there be sixteen such sections counted for each field, it is necessary to count the finest particles in but one and the next finest in but four and multiply the counts by sixteen and four, respectively. To make this impersonal, the particular sections should be selected in advance and the same ones counted whether or not they appear to represent the general condition of the field. In these sizes, the irregularities disappear quickly with a count of several fields.

The microscopic count, first applied to a narrow range of size, may with good microscopic and statistical procedures be made to cover a still wider range of diameters. It may be used with the paint pigments, mineral fillers, cement, pulverized coal, and, in fact, powders of homogeneous materials of less than sieve size (75 to 100 microns) down to a minimum determined by the wave length of the light (quarter micron with white light).

Combination Methods:

Each of the direct methods of sizing is limited by the restricted range of diameters over which it is applicable. This makes it difficult to determine with exactness the fineness distribution curve of material containing both coarse and fine particles. It is reasonable to expect that combinations of these methods will extend the working range. Inasmuch as the dimension which is measured is different for each method, it is necessary to investigate the difference and to determine the requisite factors for bringing together into a continuous whole the differently sized portions of the fineness distribution curve.

The special ranges of size to which each of the direct methods is applicable are summarized here from the previous discussion.

1. Sieving is rapid and accurate to as low as 74 microns. Separation is determined by the fitting of the minimum cross-section of the particle through the sieve aperture.

2. Elutriation has its greatest usefulness from 5 to 100 microns. Within that interval separation is supposed to follow Stokes' law. The diameter may be calculated as that of a spherical particle which has the same settling velocity as the actual particle, or it may be measured microscopically.

3. Microscopic sizing has been developed for the range of a few tenths of a micron to about 10 or 20 microns; but, with care, it may be used up to 100 microns or more. The particle diameter may be any of several; but the least diameter is usually missed.

The literature contains mention of some combinations of these methods. Weigel(7) used elutriation and microscopic sizing over the range 0 to 100 microns for mineral fillers, while Klein and Parrish(10) have used it for paint pigments. Gaudin(11) has used sieving and elutriation over the wide range of size of pulverized solids in which he was interested. Over a similar range, the author has used a combination of sieving and elutriation with microscopic sizing of the fractions or a less laborious procedure combining sieving and direct microscopic sizing of the sub-sieve material.

Within the range of sieve sizes, other methods than sieving are seldom applied. In the finer sizes, elutriation is not ordinarily regarded as a safe test because of the many difficulties in the test and some uncertainty as to the accuracy of Stokes' law for irregular particles. It is common practice to separate the material into fractions by elutriation, with subsequent determination of the diameters microscopically. The important combinations of sizing methods are then:

1. Sieve - elutriation - microscope
2. Sieve - microscope
3. Elutriation - microscope.

The combination of sieve - elutriation - microscope presents the problems of sizing which are common to the others. In work over that range, the author has utilized the following procedure for ground sand. It is applicable to homogeneous minerals and other ground products.

A weighed sample of dry material (10 to 100 g.) is dispersed in water and washed through a No. 325 sieve. When the separation appears to be complete, the wet sieve containing the oversize is placed over a clean sieve bottom and then into the oven. All of the dried oversize, including a few grains which pass through the sieve on drying, is recovered and sieved on a standard nest. Material passing the No. 200 sieve is added to the sieve washings and the entire mass is evaporated to dryness. The dry lumps are broken and made homogeneous by tumbling them with a few pebbles in a small pebble mill. This is sub-divided into fractions by elutriation or sized directly with the microscope. Three or four fractions and the overflow are sufficient. The author uses liquid elutriation, the equipment consisting of different sized vessels through which the liquid moves in a continuous series flow. The overflow from the last stage is filtered through paper and the fine particles in it recovered by ignition of the paper. At the completion of the run the contents of each elutriation vessel are filtered through alundum crucibles, dried and weighed. Once a crucible is prepared for a given stage of elutriation by a preliminary run, the solid may be recovered quantitatively. Each fraction is weighed in the crucible, and the crucible is weighed before filtration and after cleaning. The difference between the first two values is the weight of the fraction; the last value is a check upon the recovery and cleaning. Each fraction is carefully stirred to make it homogeneous; slides are prepared and sized with the microscope. In addition, the material on the No. 200 sieve is also sized microscopically to give a factor for correlating sieve mesh and microscopic diameters. The plot of percentage per micron *versus* diameter for each fraction is made upon graph paper and the summation of the ordinates for the different fractions at each diameter constitutes the curve for the total sample.

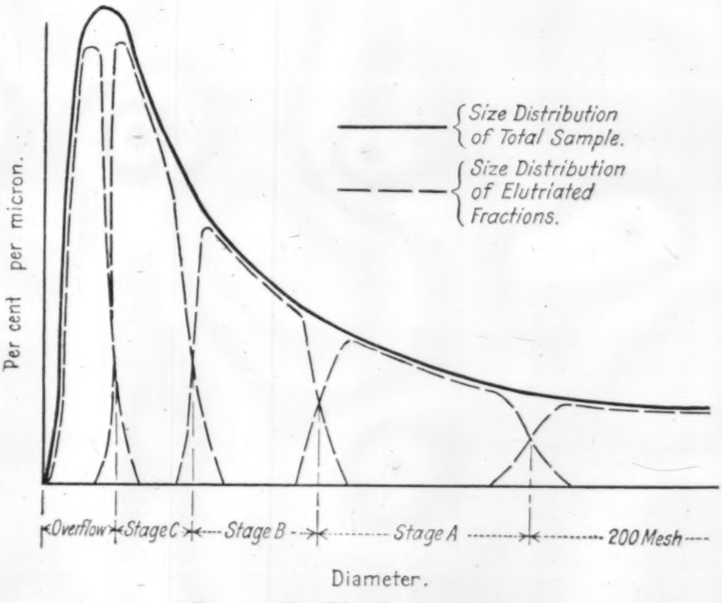

FIG. 4.—Size Distribution Curves.

The size distribution curves overlap from one fraction into another, as illustrated in Fig. 4. This is due to failure of the elutriation equipment to effect complete separation and to the non-uniformity of the particles which gives different microscopic diameters to particles having the same settling velocity.

In view of the great changes in slope at the fringes, narrow size intervals must be used for accurate determination. The shape of the curve is determined by the results between the fringes. It is therefore essential that there be a sufficient number of size intervals in that region to determine the form and height of that portion of the curve.

On the same grounds, the elutriation intervals should not be too close, else the overlapping area is too great a portion of the entire curve. That is why three or four stages of elutriation with recovered overflow are sufficient for the sub-sieve sizes.

In some cases, a less elaborate procedure is used. The elutriation is carefully performed and the microscope is used to estimate the limiting diameters of the fractions. With good judgment, these limits are determinable, although overlapping of the ranges creates some uncertainty. The average of the lower limit of one fraction and the upper limit of the next finer fraction, if they are not greatly different, gives a satisfactory diameter. With air elutriation the values cannot be determined in that manner as the fines are carried away.

TABLE III.—MICROSCOPIC AND SIEVE SIZING OF GROUND SILICA.

Diameter, microns	Percentage per Micron (Sample 11)		Percentage per Micron (Sample 13)	
	Microscope	Sieve	Microscope	Sieve
900	0.00	....	0.00	0.00
850	0.00	....	0.09	0.00
800	0.00	....	0.19	0.03
750	0.00	0.00	0.30	0.10
700	0.01	0.00	0.34	0.26
650	0.02	0.00	0.25	0.38
600	0.03	0.005	0.12	0.33
550	0.04	0.01	0.085	0.19
500	0.05	0.02	0.08	0.11
450	0.07	0.035	0.07	0.09
400	0.09	0.06	0.06	0.085
350	0.12	0.09	0.06	0.08
300	0.15	0.12	0.06	0.08
250	0.18	0.17	0.06	0.075
200	0.20	0.22	0.06	0.075
150	0.22	0.27	0.055	0.07
100	0.28	0.35	0.05	0.07

Pearson and Sligh(5) collect particles of the last dust to come off at any one air velocity. As these are the coarsest recovered at that velocity, the diameter of separation is determinable.

Attention has been directed toward a correlation factor for sieve and microscopic scales. Microscopic sizing over the entire sieve range gives a fineness distribution curve based on the same criterion of diameter. However, if the particles are statistically similar in form on each sieve, the determination of the microscopic diameter corresponding to the No. 200 sieve gives a correlating factor which will make the curves meet. The microscopic scale is then calibrated so that the microscopic diameter found for 200-mesh material is set equal to 74 microns, the sieve aperture.

The part that particle shape plays may be seen in Table III. The microscopic diameter for 200 mesh has been determined as 100 microns, which is slightly less than the diagonal of the sieve aperture.

Corresponding points in the microscopic curve occur at a diameter which is higher than for the sieve curve. If, however, the microscopic diameters are reduced in the ratio of 74 to 100 and the corresponding

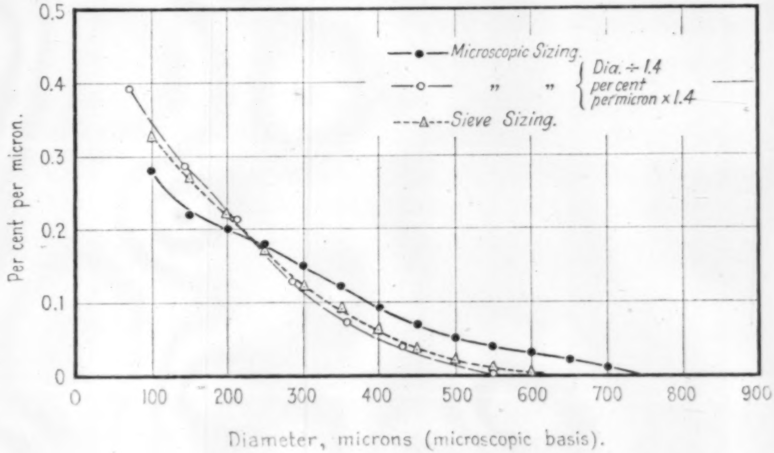

FIG. 5.—Comparison of Microscopic and Sieve Sizing Data of Sample No. 11. Feed, 1100 g., 24 to 30 mesh. Ground in laboratory jar mill, 2 hours.

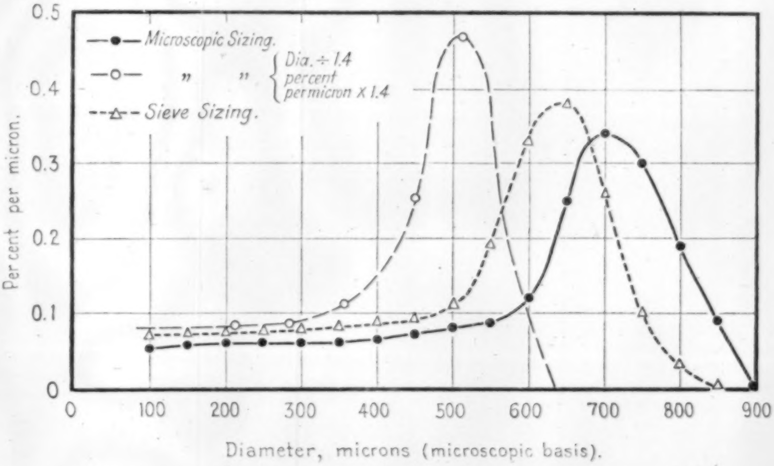

FIG. 6.—Comparison of Microscopic and Sieve Sizing Data of Sample No. 13. Feed, 1100 g., 24 to 30 mesh. Ground in laboratory jar mill, 30 minutes.

per cent per micron values increased by the same ratio, the curves are alike (Fig. 5). On the other hand, Sample 13 shows marked variation in the coarse range (Fig. 6). The microscope and the sieves are measuring unrelated functions because of a marked change in particle shape between the original mill feed and the product of grinding.

Further evidence has been brought to bear to establish the extent of the variation of particle shape. A portion of each sieve fraction was weighed and every particle was counted. The fractions were closely limited in diameter. $x + \Delta x$ was not materially greater than x and also the ratio of the two was constant. Therefore, all particles in a range may be considered the same in size. From the number of particles, the specific gravity of the material, and the weight of sample, the diameter of a sphere having the average weight of a particle may be calculated. That value is called the mean spherical diameter. It bears a certain relation to the average sieve aperture which differs with the irregularities in the particles. (See Table IV and Fig. 7.) This is offered as the means of measuring a roundness or shape factor for closely sized material and as supplementary evidence to show that correlation of two sizing scales is dependent, at least at the point of overlapping, upon the shape of the particles.

TABLE IV.—COMPARISON OF MEAN SPHERICAL AND SIEVE DIAMETERS.

Sieve Aperture, mm.	Average Diameter, mm.	Retained on Sieve, per cent	Average Weight of Particles, mg.	Mean Spherical Diameter, mm.	Mean Spherical Diameter divided by Average Sieve Opening
0.701					
0.589	0.645	5.4	0.560	0.606	0.938
0.495	0.542	3.9	0.293	0.488	0.900
0.417	0.456	7.0	0.164	0.403	0.884
0.351	0.384	8.1	0.103	0.345	0.898
0.295	0.323	9.1	0.069	0.301	0.932
0.247	0.271	6.5	0.043	0.257	0.948
0.208	0.227	7.0	0.0236	0.215	0.947
0.175	0.191	6.7	0.0147	0.180	0.944

REPRESENTATION OF DATA

In presenting fineness data the following points should be stressed:

1. *Simplicity.*—It is desirable to record with a minimum number of figures the significant features about the fineness of a material.

2. *Usefulness.*—The values which represent the fineness should be directly significant for the ultimate purpose in mind when the test was made.

3. *Fundamental Value.*—It is preferable to report values in terms of standard units, rather than empirical ones. Results from different laboratories are then comparable.

4. *Universal Application.*—Since the fineness determination may be useful as a measure of many functions, as surface, average diame-

ter, maximum size, the values should be directly and simply transferable to the significant value.

The interests of simplicity and usefulness are frequently served by one or two fineness constants. More exacting requirements of fundamental value and of universal application demand the fineness distribution curve.

Constants are valuable when they measure with one number a function which is the important variable with particle size. For instance, Abrams(12) utilizes the fineness modulus to define the sizes of sand and stone for concrete aggregates. Others have used average size and uniformity of size constants for sands. Where rate of chemical reaction is important or where coarse grits may be objectionable in product, the percentage of material coarser than a specified sieve is the significant constant. Green, Perrott and Kinney, Weigel and others have utilized average diameters to represent the fineness of paint pigments and mineral fillers. The several bases of calculation are summarized by Green(13). The total surface per unit weight or per unit volume is used to represent a factor which is commonly conceded to measure the chemical activity of a powder. Recent efforts by Sheppard and Lambert(9), Martin(8), Gaudin(11) and the author to

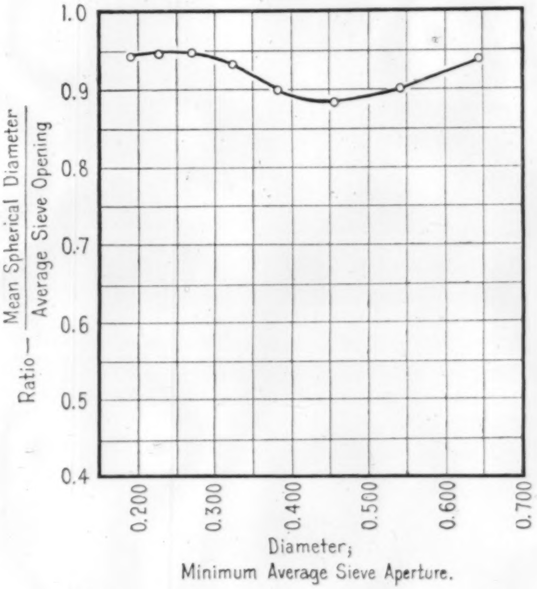

FIG. 7.—Comparison of Mean Spherical and Sieve Diameters of Sample No. 3.

develop statistical relations for fineness curves may lead to constants of greater universal application in that there may be derived the entire fineness distribution curve and the usual fineness constants by mathematical calculation from the general relations and these constants.

Complete presentation of the data is to be had with the fineness distribution curve. It shows the frequencies, weights, or other functions with respect to the diameters. From it may be calculated the significant constants for fineness comparison.

Selection of the variables is a most important consideration. Fortunately the abscissa is invariably a unit of particle diameter. It may be expressed in inches, millimeters, or microns, the last named being particularly desirable for sub-sieve sizes. The diameters will differ somewhat with the bases of sizing. Ordinates are usually weights or frequencies. The following are important bases of representation and they typify the more fundamental schemes.

1. Particle frequency per unit diameter per unit weight of sample *versus* diameter.
2. Cumulative percentage by weight *versus* diameter.
3. Percentage by weight retained on sieve *versus* diameter.
4. Percentage by weight per unit diameter *versus* diameter.

The actual values upon four samples of silica ground in the laboratory are submitted for the purpose of comparing these bases in their several forms.

Particle Frequency per Unit Diameter versus Diameter.—The frequency basis has been most largely used in the range of colloidal sizes and paint pigments. It seems to have no application to particles coarser than sieve sizes. Ordinarily the values of frequency represent an actual count under the microscope. Any reasonable total number of particles is recorded. The data of the British Portland Cement Association is submitted in that form. Green uses frequency for paint pigments and hesitates to calculate from it to the weight basis.

The actual count is dependent upon the density of the slide and the number of fields counted at any magnification and bears no relation to counts from other slides of the same or different samples. Inasmuch as it is somewhat more fundamental to compare frequencies resulting from the grinding of a specified weight of sample, the values given in Table V have been calculated to and from the weight basis and are not the actual count.

The plot of frequency per unit diameter against diameter serves well in the short ranges of paint pigment sizes. Over a greater range of diameter, the logarithm of this frequency value is essential, as the

TABLE V.—FREQUENCY PER MICRON PER UNIT WEIGHT OF SAMPLE: GROUND SILICA.

Diameter Range Number	Average Diameter, micron	Frequency per Micron per Unit Weight			
		Sample 13	Sample 105-W	Sample 304-O	Sample 7
26	767	0.000176	0.00000044	....	....
25	645	0.00145	0.00000186	....	....
24	542	0.000718	0.00000627	....	....
23	456	0.000835	0.0000169	....	....
22	384	0.00121	0.0000546	....	....
21	323	0.00222	0.000281	0.000059	....
20	271	0.00298	0.00120	0.000100	....
19	227	0.00607	0.00715	0.000341	....
18	192	0.00912	0.0217	0.000843	....
17	161	0.0190	0.093	0.00287	....
16	136	0.0195	0.122	0.00555	....
15	114	0.0537	0.410	0.0302	....
14	96	0.0306	0.602	0.0777	....
13	81	0.0695	0.698	0.1726	....
12	64.75	0.2025	2.260	2.49	....
11	46.25	0.509	5.34	10.06	1.932
10	32.38	2.29	15.04	31.13	25.30
9	23.13	6.78	63.4	124.2	138.3
8	16.19	20.0	221	546	724
7	11.56	46.2	400	1605	1553
6	8.09	100.3	1045	4970	6320
5	5.78	294	2570	8860	16300
4	4.05	1440	7890	27000	81200
3	2.89	4300	16300	65700	255000
2+	2.02	20700	45600	270000	1232000
2−	1.44	53000	121000	624000	3680000
1+	0.87	281000	294000	2320000	7680000
1−	0.289	706000	2650000	7050000	69800000

TABLE VI.—LOGARITHMS OF FREQUENCIES IN TABLE V.

Diameter Range Number	Average Diameter, micron	Log. Frequency per Micron per Unit Weight			
		Sample 13	Sample 105-W	Sample 304-O	Sample 7
26	767	3.755	6.357	.....	.....
25	645	2.839	5.731	.....	.....
24	542	3.144	5.203	.....	.....
23	456	3.078	4.772	.....	.....
22	384	2.917	4.263	.....	.....
21	323	2.653	3.551	4.229	.....
20	271	2.526	2.921	4.000	.....
19	227	2.217	2.145	3.467	.....
18	192	2.040	1.653	3.074	.....
17	161	1.721	1.031	2.542	.....
16	136	1.710	0.914	2.256	.....
15	114	1.270	0.387	1.520	.....
14	96	1.094	0.220	1.110	.....
13	81	1.158	0.156	0.763	.....
12	64.75	0.694	0.354	0.396	.....
11	46.25	0.293	0.728	1.002	0.263
10	32.38	0.360	1.177	1.493	1.403
9	23.13	0.831	1.802	2.093	2.141
8	16.19	1.301	2.344	2.737	2.860
7	11.56	1.665	2.602	3.205	3.191
6	8.09	2.001	3.019	3.696	3.801
5	5.78	2.468	3.756	3.947	4.212
4	4.05	3.158	3.897	4.432	4.910
3	2.89	3.633	4.212	4.818	5.407
2+	2.02	4.316	4.659	5.431	6.091
2−	1.44	4.725	5.083	5.795	6.566
1+	0.87	5.449	5.468	6.365	6.885
1−	0.289	5.894	6.423	6.848	7.844

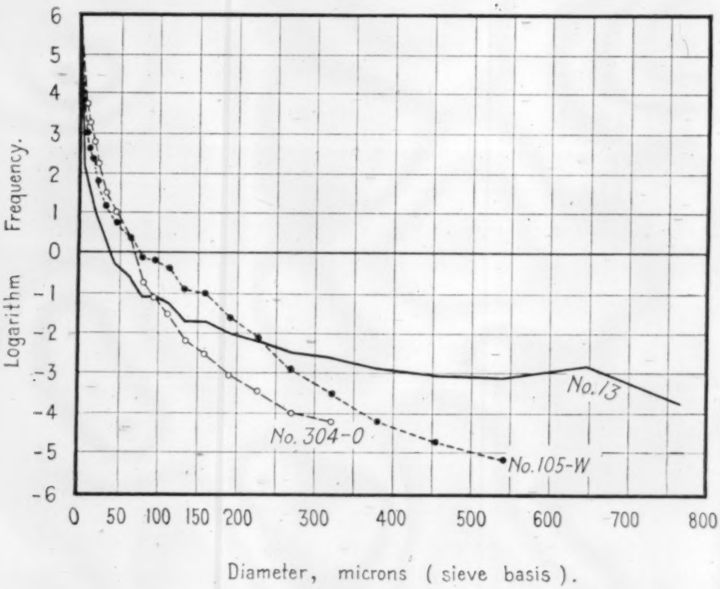

FIG. 8.—Frequency Curves, Type I. Logarithm of particle frequency per micron based on unit weight *versus* particle diameter.

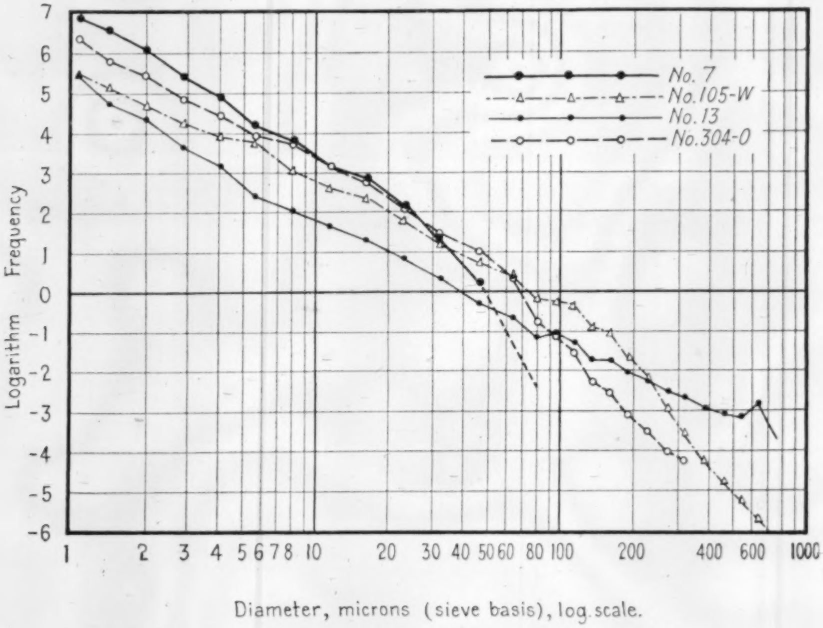

FIG. 9.—Frequency Curves, Type II. Logarithm of frequency *versus* logarithm of diameter.

frequencies in the finer sizes are of a different order of magnitude from those in the coarser sizes. The frequencies have been calculated directly into their respective logarithms, values of which are given in Table VI.

Values for three of these curves have been represented graphically in Fig. 8.

Another graph, Fig. 9, shows the data on the four samples, being less crowded in the fine ranges.

Cumulative Percentage by Weight versus Diameter.—In the coarse range of sizes, the common data is that of sieve sizing. Standard sieve mesh have already been discussed as well as the factors bearing on sieve sizing. Of the several methods of representation the cumulative percentage basis is most common. The data becomes a series of diameters or sieve apertures with the total percentage by weight either greater than or less than each sieve aperture. This is very frequently expressed as a cumulative percentage either undersize or oversize. Such a curve serves to smooth out the defects of any individual sieve. It is independent of the apertures of the sieves used in test. When properly correlated with this, data of material finer than sieve sizes may be added to the sieve data. Results of test runs used for comparison are given in Table VII.

TABLE VII.—CUMULATIVE PERCENTAGE OVERSIZE *versus* DIAMETER: GROUND SILICA.

	Diameter, micron	Cumulative Percentage Oversize			
		Sample 13	Sample 105-W	Sample 304-O	Sample 7
Sieve...............	701	10.55	0.03	....	....
	589	54.50	0.09	....	....
	495	65.32	0.19	....	....
	417	71.53	0.32	....	....
	351	76.10	0.52	....	....
	295	80.34	1.05	0.11	....
	246	83.28	2.24	0.18	....
	208	86.01	5.47	0.33	....
	175	88.17	10.61	0.53	....
	147	90.41	21.60	0.87	....
	124	91.56	28.76	1.19	....
	104	93.16	41.11	2.11	0.18
	88	94.32	49.74	3.24	0.25
	74	94.84	55.01	4.55	0.32
Microscope.........	55.5	95.88	66.50	17.13	0.32
	37	96.83	76.30	35.45	3.65
	27.75	97.58	81.08	45.29	11.63
	18.5	98.37	88.43	59.61	27.49
	13.875	98.78	92.83	70.38	41.74
	9.25	99.12	95.74	81.93	52.87
	6.94	99.24	97.05	88.01	60.59
	4.625	99.37	98.23	91.95	67.93
	3.469	99.48	98.85	94.03	74.18
	2.313	99.60	99.32	95.90	81.31
	1.734	99.70	99.53	97.17	87.20
	1.156	99.80	99.75	98.24	93.60
	0.578	99.99	99.96	99.90	99.03
	0.000	100.00	100.00	100.00	100.00

The data given in Table VII is represented graphically in Fig. 10. It might equally well be used as the cumulative percentage undersize, and the curve would simply be inverted. To eliminate the congestion of points in the fine sizes the diameter may be plotted logarithmically as in Fig. 11. To make simple certain energy calculations, there may be plotted in place of the diameter its reciprocal. The form of such curves is not unlike that of Fig. 11.

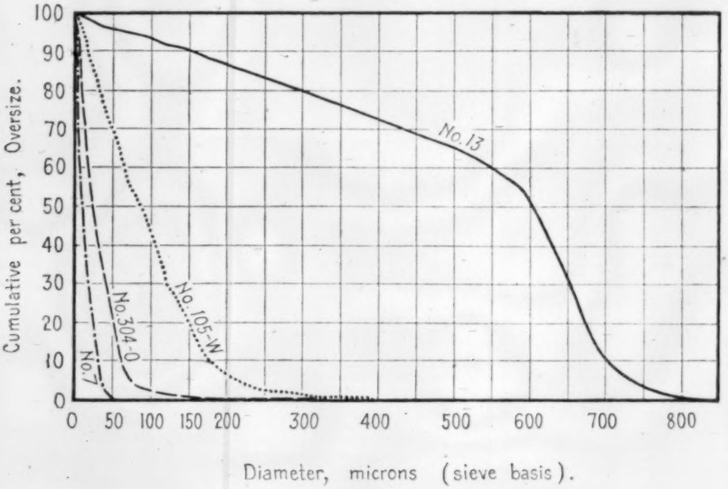

FIG. 10.—Cumulative Percent Curves, Type I.

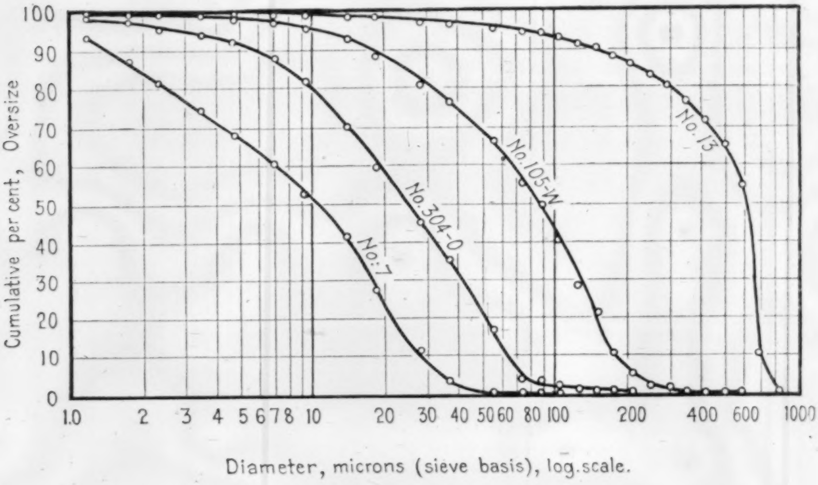

FIG. 11.—Cumulative Percent Curves, Type II.

In these curves, the data are welded into an integrated whole; individual values are lost. The slope of the curve and inflection points are significant. The method has its greatest use in bringing to a common basis and making available for simple comparison of fineness and uniformity the data obtained with differing sieve scales.

Percentage by Weight on Sieve versus Sieve Diameter.—Along with the cumulative percentage plot, this method for representing data has been used in the sieve ranges of size. The direct sieve data as well as some correlated values in the range of microscopic sizing are given in Table VIII.

TABLE VIII.—PERCENTAGE RETAINED ON SIEVE *versus* DIAMETER: GROUND SILICA.

Sieve Aperture, micron	Retained on Sieve, per cent			
	Sample 13	Sample 105-W	Sample 304-O	Sample 7
701	10.50	0.03	....	....
589	43.71	0.06	....	....
495	10.77	0.10	....	....
417	6.18	0.13	....	....
351	4.54	0.20	....	....
295	4.21	0.53	0.11	....
246	2.92	1.17	0.07	....
208	2.71	3.19	0.15	....
175	2.14	5.09	0.20	....
147	2.22	10.89	0.33	....
124	1.14	7.09	0.31	....
104	1.60	12.23	0.90	0.17
88	1.15	8.54	1.10	0.06
74	0.52	5.22	1.28	0.06
52	0.48	5.26	7.70	0.83
26	0.38	3.00	6.02	5.96
13.05	0.18	1.81	5.56	6.35
6.52	0.06	0.61	2.52	3.77
3.26	0.06	0.27	0.98	3.35
1.63	0.05	0.11	0.59	3.07
0.81	0.05	0.05	0.41	1.33

The actual percentages upon each sieve depend upon the size of opening in the next larger sieve. It is thus necessary to specify the nest of sieves used. Values obtained with a Tyler nest in which the sieve interval varies by the fourth root of two or by the square root of two are best plotted with logarithm of sieve aperture as abscissa, thus obtaining uniform intervals between each point. Such a curve has a peak with sides receding more slowly toward zero aperture than toward larger sizes. (See Fig. 12.)

It is to be observed that these curves contain irregularities in their individual values. They are used to the full limit of their accuracy, whereas, in the two previous methods, the individual values were made less conspicuous beneath the cumulative effect of the whole or on account of the logarithmic scale which was necessitated by rapid changes in the order of magnitude of the values. For the most exact comparisons, the values should be used to the limit of accuracy and the curve should be smoothed to correct the errors in individual values. Then, the errors in one value are absorbed at adjacent points which is the most probable condition.

Percentage by Weight per Unit Interval of Diameter versus Diameter.—In the sub-sieve sizes, representation of weight has been frequently referred to ranges of particle diameters. The sizing intervals have been so different and so irregular that there has been little tendency to standardize on a graduated interval analogous to the sieve scales in which the intervals vary in a geometric series. Much of this data is presented in terms of the interval in which it was sized. To eliminate the irregularities in weight due to the different intervals and to give fundamental value to the method of representation, the weight per unit interval of diameter has been used. This usually takes the form of percentage per micron interval and is plotted against the diameter. The percentage in a millimicron (0.001 micron) for colloids has been proposed.

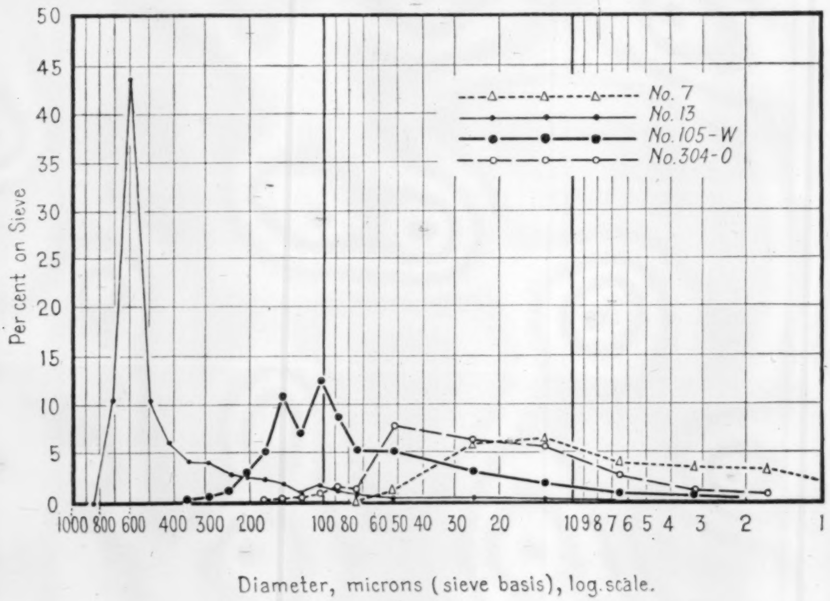

FIG. 12.—Percentage on Sieve. Sieve variation based on $\sqrt[4]{2}$.

Here, too, the values are used to the full limit of accuracy and individual points may be improved by smoothing. The values may be plotted as histograms or blocks having the width of the interval sized and a height equal to the average percentage per micron over that interval (Figs. 14 and 15). A smooth curve may be drawn which

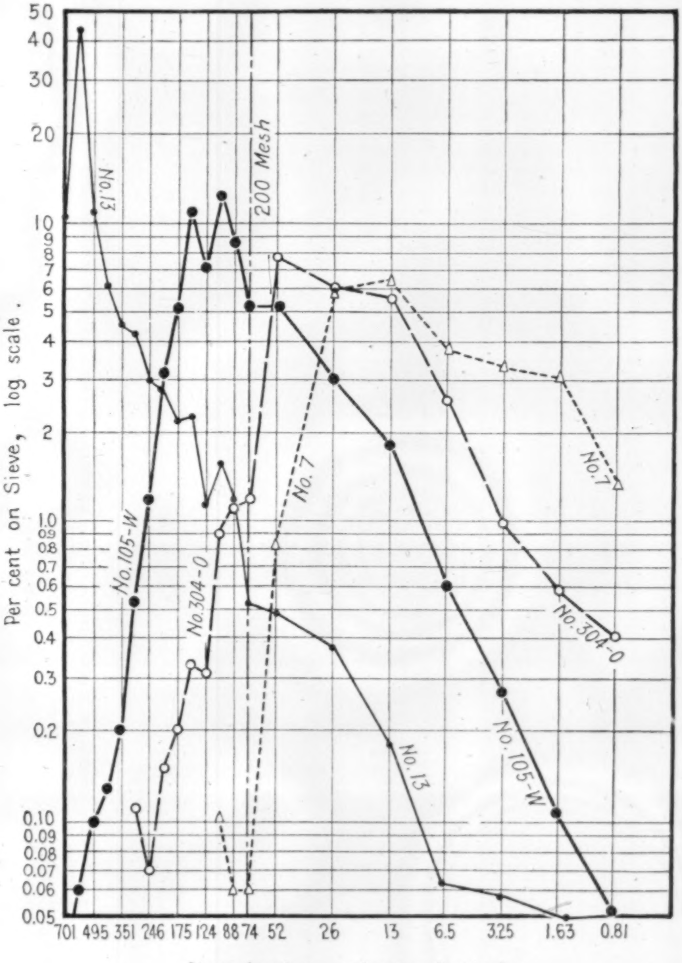

FIG. 13.—Relation of Logarithm of Percent on Sieve to Sieve Aperture. Class intervals function of $\sqrt[4]{2}$.

will include and exclude equal areas of these blocks in the same region. As an alternative, the average percentage per micron and the average diameter of the interval may be represented. A moving average of three points will smooth out many individual errors. More than three points tends to flatten the curve and eliminate significant peaks. The author has used both independently and then combined their forms into one. There is usually but little difference in these two results.

Of the weight bases, this seems the preferable type, as it has fundamental value, is independent of the intervals selected providing a sufficient number are taken, gives directly a true impression of fineness, and has universal application in making other calculations and in covering both coarse and fine ranges of size.

TABLE IX.—PERCENTAGE BY WEIGHT PER MICRON *versus* DIAMETER: GROUND SILICA.

Diameter Range Number	Diameter Range, micron	Percentage per Micron			
		Sample 13	Sample 105-W	Sample 304-O	Sample 7
26	833 to 701	0.080	0.0002	....	....
25	701 to 589	0.390	0.0005	....	....
24	589 to 495	0.115	0.0010	....	....
23	495 to 417	0.079	0.0016	....	....
22	417 to 351	0.069	0.0031	....	....
21	351 to 295	0.075	0.0095	0.002	....
20	295 to 246	0.060	0.024	0.002	....
19	246 to 208	0.071	0.084	0.004	....
18	208 to 175	0.065	0.154	0.006	....
17	175 to 147	0.079	0.389	0.012	....
16	147 to 124	0.049	0.308	0.014	....
15	124 to 104	0.080	0.612	0.045	0.009
14	104 to 88	0.072	0.534	0.069	0.004
13	88 to 74	0.037	0.373	0.092	0.004
12	74 to 55.5	0.055	0.614	0.678	0.000
11	55.5 to 37	0.050	0.524	0.987	0.180
10	37 to 27.75	0.078	0.512	1.06	0.862
9	27.75 to 18.5	0.084	0.786	1.54	1.714
8	18.5 to 13.88	0.085	0.941	2.32	3.08
7	13.88 to 9.25	0.072	0.620	2.49	2.41
6	9.25 to 6.94	0.053	0.553	2.63	3.34
5	6.94 to 4.63	0.057	0.498	1.72	3.17
4	4.63 to 3.47	0.096	0.526	1.80	5.41
3	3.47 to 2.313	0.104	0.395	1.59	6.17
2+	2.313 to 1.734	0.171	0.377	2.23	10.18
2−	1.734 to 1.156	0.159	0.364	1.89	11.06
1+	1.156 to 0.578	0.344	0.360	2.84	9.40
1−	0.578 to 0.000	0.017	0.064	0.17	1.68

INDIRECT METHODS OF SIZE DETERMINATION

On account of the great time required to make direct and absolute measurements, they have not been appreciated nor utilized to the fullest extent by the industries. The indirect methods have been developed primarily for rapid analysis.

Normally, these methods are based upon a function the exact variation of which to distribution of particle size is still uncertain. They are more directly a measure of that variable itself and are quite frequently more useful in that respect than the fineness distribution curve.

Settling Rate in a Fluid Medium:

It has already been noted that elutriation may be used for direct sizing. The separation may be inexact on account of the many factors upon which it is dependent. However, the method has been called direct, because the diameter of the fractions need only be checked with the microscope. The same principles of settling are used as an indirect test without separation of the material into recoverable fractions.

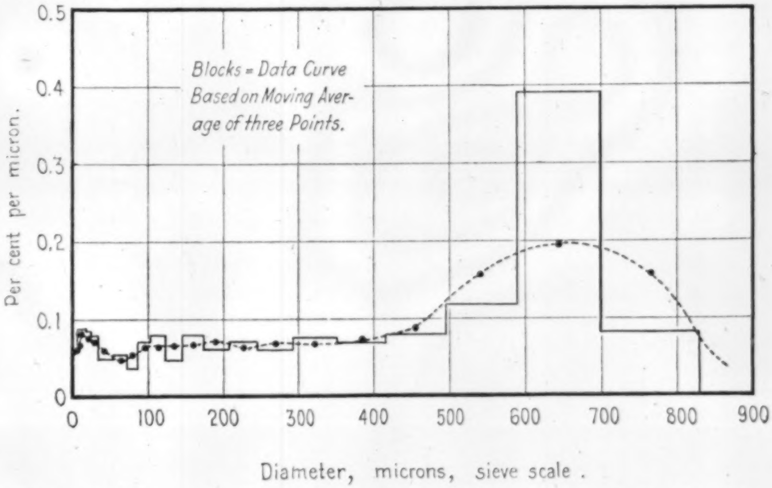

FIG. 14.—Relation of Percentage per micron to Diameter, Sample No. 13–1000 g. of Sand Ground Dry ½ Hour.

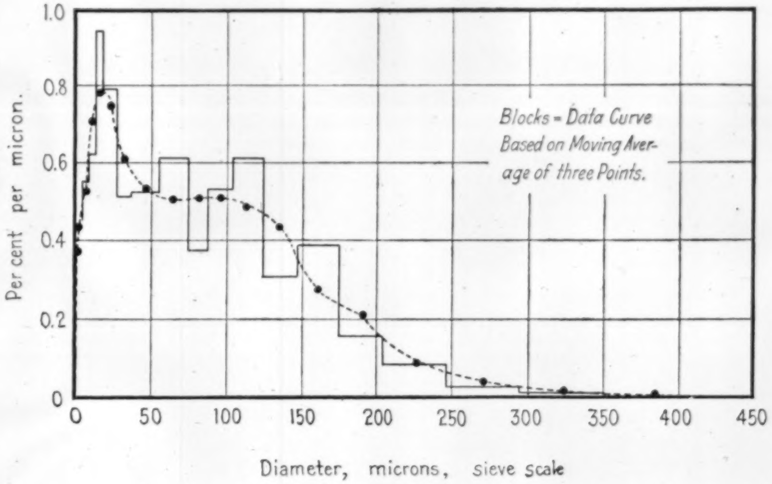

FIG. 15.—Relation of Percentage per Micron to Diameter, No. 105-W, 400 g. of Sand Ground Wet 1 Hour.

This test depends directly upon the fundamental laws of settling, and careful manipulation of technique.

In its several forms, the test is essentially a method for determining either the weight of solid settled from a fluid in a given time or the variation in concentration of solids in the fluid with time. Calbeck and Harner(17) have measured the weight of solid which collects upon a tared balance pan suspended at a definite level in the fluid. Several others have used a differential manometer to measure the density of the column of fluid as the particles settle past a reference plane.

Oden(14), Svedberg(15), Kelly(16) and Calbeck and Harner(17) have discussed the means of interpreting and analyzing the time settling curve which results from any of these tests. Without further explanation, the method is this: The relation between the diameter of a spherical particle and its time of settling through the fluid height in the apparatus is calculated from Stokes' law. A series of diameters is selected and the corresponding time is calculated. These values are noted on the curve of time versus per cent settled. Tangents to the curve are drawn at these points and their intercepts in terms of the percentage settled represent the amounts falling between the corresponding diameters. The distribution curve is obtained by calculation.

A method for finer particles has been proposed by Svedberg(18). In this, the centrifugal action upon the mass of the particle is balanced by the colloidal forces tending to disperse the particles uniformly through the fluid.

To properly evaluate the settling test, it is necessary to know its possible limitations. The fluid viscosity must be constant, requiring a close control of temperature. The particles are assumed to be spherical, but an "equivalent diameter" for irregular particles may be assumed. The smaller particles may be affected by electrostatic and surface tension effects which make it impossible to apply Stokes' law. The fundamental law of this test supposes the independent settling of a particle, and this holds only in relatively dilute suspensions. If the solids are so dispersed, the accuracy of the observation is impaired. When the solids are too concentrated, the laws of hindered settling apply. Above all, the solid must be completely dispersed in the fluid and there must be no agglomeration.

There is but little evidence to show how this method checks with the frequency count. The distribution curves obtained from settling data are not always in accord with the expected nature of things. It does not seem reasonable that a mixture of particles will be discontinuous in size, that is, some particles of one diameter, none of the next, some more of the next, etc. These variations are due to inaccuracies in the curve form or in drawing the tangents. It is probable that they can safely be eliminated by smoothing procedures. It remains for a group such as the Technical Committee on Size and Shape of the Society's Committee E-1 on Methods of Testing to compare distribution curves obtained by the several methods and to determine the value of the time-settling test.

The method has been used for the coarser paint pigments, and other products of the same range of size, namely, about 2 to 30 microns. Coarser particles settle too rapidly to be recorded while finer ones settle too slowly for accuracy in drawing tangents.

Apparent Density:

The test of apparent density has been used as a measure of bulking value for coarse materials like sand as well as for the finer materials like cement, mineral fillers, and the pigments. That the values so obtained have a bearing upon fineness makes it of interest in this connection.

TABLE X.—VARIATION IN APPARENT DENSITY WITH TIME OF GRINDING.

Sample	Time of Grinding hr.	Time of Grinding min.	Revolutions	Apparent Density
Unground Sand	0	0	..	1.710
No. 13	0	30	1 756	1.805
No. 12	1	4	3 873	1.655
No. 11	2	0	7 219	1.580
No. 8	5	9	21 166	1.430
No. 7	14	7	55 143	1.205

Very little can be said as to the exact relation between fineness and apparent density. It is known that materials of uniform size bulk more loosely, while those containing a mixture of sizes bulk more compactly. The presence of moisture in the sample materially affects the apparent density, as it prevents close packing of the mass and adds a weight of water. The behavior of pulverized material under this test may be shown by data, given in Table X, on silica which has been ground in a ball mill. It was obtained by tapping a graduate containing a known weight of the sample until it settled to a minimum volume.

The apparent density becomes greater with the formation of the first fine material. After a certain quantity of fines have been formed the apparent density decreases with further grinding. The value may then become a criterion of fineness for certain materials and under defined conditions of test. It should be remembered that the apparent density is not a measure of the distribution of particle size and that the same results in density may be had with quite different curves of fineness distribution.

There has been some tendency to give as "fineness modulus" the ratio of actual and apparent densities. In theory, that would make comparable the data on different materials. The inference is as yet unverified and extremely doubtful.

The test of apparent density is of value in estimating particle size when it is used under closely controlled conditions. It is of infinitely more value in being a direct test of "packing" characteristics, and is more rapid and accurate than any estimate of "packing" which might be made from the fineness curve.

Light Dispersing Power:

Under the heading of light dispersing power, there are several tests which are criteria of fineness. They are more directly a measure of hiding or covering power and are of particular interest for paint pigments. Such methods depend upon the dispersing effect of light in passing between fluid and solid, thus varying with the indices of refraction of the fluid and the solid and with the fineness and surface form of the solid. Reduction in size of the solid gives a greater projected area when a specified amount is suspended in a fluid; and it is likely that the hiding power is in proportion to this projected area.

The tests take the following forms:

1. Direct painting of thin layers of the regular fluid and pigment, until the last layer gives when dry complete coverage of the background(19).
2. Measurement of the thickness of paste just needed to obscure a dark background.
3. Observation of the height of column just needed to obscure a light image. Adaptation of the turbidimeter.
4. Measurement by matching intensity of the light after dispersion or absorption in its passage through given chamber.
5. Measurement of the intensity of a Tyndall beam passed through a suspension of the particles.

The first two are practical tests which are applicable to paints. The others are used with dilute suspensions. Tolman(21) has found that the Tyndall-meter gives readings proportional to concentration of suspended solid, when the same material is used at different dilutions. Stutz and Pfund(20) have measured the reduction in intensity of illumination with red light and with green light using the same samples. The difference indicates that the hiding characteristics of particles approaching light wave lengths differ with the color of the light used. This confirms a more general observation that particles of pigments may be too fine to diffract the light wave.

The values obtained in these tests are reported in relative units or in absolute units for the specified conditions of test. They are usually quite accurate, if being easily duplicated and giving continuously varying curve forms is an indication of this. At present, it has not been established what size function these values measure. However, the greater hiding power means greater fineness until the particles approach in size the wave length of light. The author has secured some evidence on powdered coal, which is opaque, that confirms a theory that the turbidimeter measures projected area of the particles, and that in turn is a function of surface. If this can be verified for many kinds of materials, the values will have great significance.

Rate of Solution:

Where it is possible to find a powder of homogeneous material and a solution which will slowly dissolve it, the rate of solution becomes a measure of the fineness. Wolff(22) has measured the fineness of glass powder, comparing the amount dissolved in a mixture of sodium carbonate and caustic soda with that dissolved from a known surface of glass plate. Kriege(23) has used the dissolution method for determining the surface of limestone. Martin(24) has also used such a method for quartz. Parallel with this, Gross and Zimmerly(25) have made a similar investigation.

In most of the work, it has been assumed that the rate of solution is independent of the size of the particle and is directly a measure of surface. Gross and Zimmerly have recognized the probable errors resulting from the progressive elimination of fine particles, from the reduction of surface as the particles are reduced in size and from the change in concentration of the solution with time. They determine the amounts of quartz dissolved at certain time intervals and extrapolate to zero time. This gives the initial rate of solution, which they claim is more closely a measure of the surface. When calibrated against a known surface of the same material a value representing the surface of the powder is obtained.

Miscellaneous Tests:

A number of other methods have been proposed, which are of special application or are limited as to present use:

1. *X-ray Dispersion.*—The dispersion of X-rays in a manner not unlike the light dispersion method has been proposed and tested in the finer sieve sizes as a means of securing an average diameter(26).

2. *Plasticity.*—The stiffness of a paste of a fluid and a powder is a function of the fineness of the powder. The test is more a measure of plasticity than of actual particle diameter(27).

3. *Test in Rubber.*—Weigand(28) has proposed the stress-strain curve for dispersions of powder in rubber. The test is more a measure of the effect of the filler in rubber than it is directly one of particle size.

4. *Absorption.*—The amount of fluid necessary to fill the interstices in a powder varies with the fineness and uniformity of size of the particles, as well as other factors. The absorption test is more a measure of the amounts of fluid and solid needed to produce a given consistency than it is a measure of fineness.

5. *Adsorption.*—The adsorption of vapors from a gas or of solids from solution might be expected to vary with the surface of powder dissolved in the fluid. A test of this type may be developed to be analogous to and have more general applicability than the rate of solution method.

CONCLUSIONS

1. Fineness tests are of two types: direct measurement and indirect comparison. By actual observation, the direct methods measure the amounts of material of each size and they should be regarded as standards of reference. By the measurement of some property which varies with fineness, the indirect methods give in a single value an estimate of the degree of fineness, but they should be checked for accuracy by the direct method. These methods are usually rapid and often are a more useful measure of the property of the material than a fineness curve obtained by direct measurement.

2. Sieving is rapid and accurate to as low as 74 microns, with certain limitations as to the nature of the material, the method of sizing, and the closeness of the nest intervals. Separation is determined by the fitting of the minimum cross-section of the particle through the sieve aperture.

3. Elutriation has its greatest usefulness from 5 to 100 microns. Within that interval, separation is supposed to follow Stokes' law. Such limitations on the practical operation of this law as particle shape, variation in fluid viscosity with temperature, completeness of deflocculation, and hindered settling effects, make the calculation of particle diameter somewhat uncertain. The points of separation may

be measured microscopically for exactness. Otherwise the diameter is calculated as that of a spherical particle which has the same settling velocity as the actual particle.

4. Microscopic sizing has been developed for the range from a few tenths of a micron to about 10 or 20 microns; but, with care, it may be used up to 100 microns or more. The method requires good microscopic technology to secure accurate magnification and sharp definition of the particle image, and it requires the observance of statistical principles in sampling the material, preparing the slide and selecting the fields for measurement. The particle diameter may be any of several; but the least diameter is usually missed.

5. Sieving, elutriation, and microscopic sizing may be combined to give fineness curves over a wider range of size and with greater ease than would be possible with one of these alone. Particle shape is an important factor in this correlation. A method for the measurement of a roundness or shape factor for closely sized materials is based upon a comparison of sieve diameter with the count of a weighed group of particles.

6. In representing fineness data, the following points should be stressed: simplicity, usefulness, fundamental value, and universal application. Among a few of the methods now used, those which best meet the requirements are frequency per unit diameter per unit weight of sample *versus* diameter and percentage by weight per unit diameter *versus* diameter. While a single constant would make for simplicity, most of these lack either fundamental value or universal application.

7. Such indirect methods as settling rate in a fluid medium, apparent density or packing volume, hiding or light dispersing power, rate of solution, X-ray dispersion, plasticity, and effect in products, such as rubber, putty, etc., are normally based upon a function the exact variation of which to distribution of particle size is still uncertain. They are more directly a measure of that variable itself and are quite frequently more useful in that respect than the fineness distribution curve.

ACKNOWLEDGMENT

The author desires to express his appreciation to Prof. Daniel D. Jackson, under whose direction the investigation was pursued, for guidance into this field and for aid during the progress of the study.

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SAMPLING OF PULVERIZED COAL FROM AIR STREAMS

By Lincoln T. Work[1]

Synopsis

There are two primary sources of error in sampling coal dust from air streams, those due to segregation and those due to pressure, velocity and resistance. The first can be tested for and guarded against by selecting a suitable cross-section for test where segregation is a minimum. Too close proximity to bends and long horizontal sections of pipe should be avoided. The effect of the second can be minimized by the use of a sampler like the tube and bag described in the Society's Tentative Method of Test for Fineness of Powdered Coal and the location of the sampling opening nearer the fan than the burner.

Further light may be shed on this problem by comparing samples taken from the air stream with the product from a large cyclone separator which collects all of the coal from the mill.

Introduction

With the increasing emphasis upon the fineness of pulverized fuel, it is important that measurements of fineness be accurate. Not the least significant phase of the problem is that of sampling. It is a relatively simple matter in the bin and feeder system to secure a satisfactory sample from a stream which is moving, especially when it is also being mixed. Samples withdrawn from the end of a screw conveyor during a period of uniform mill operation may be considered representative. On the other hand, it is most difficult in the direct-fired system to secure a satisfactory sample from a rapidly moving stream of air and coal dust. Many different procedures have been devised, but, as yet, none is universally accepted. It is the aim of this paper (1) to make note of typical sources of error in sampling coal moving in air streams, (2) to discuss the principles which determine the accuracy of any method, and (3) to propose an attack to determine what is an exact procedure.

Sources of Error in Typical Sampling Equipment

Sampling equipment and procedures are most diversified, notwithstanding the present efforts to standardize them. It is not uncommon for some operators to take samples from a cock on the side of the fuel piping. A bag is tied over the tip, the valve is opened, and the resulting sample is tested. Needless to say, the sample is found to be very fine. The coarser particles in the stream are too heavy to change their direction quickly and they pass by the opening. Possibly, at that point, a greater proportion of the coarse pieces is traveling along another portion of the pipe.

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[1] Instructor in Chemical Engineering, Columbia University, New York City.

Others, more careful of their technique, have no faith in a sample which involves but a portion of the air stream. They enclose the outlet pipe of the burner in a dust-tight bag and collect the entire sample which comes over. The author tested a sample said to have

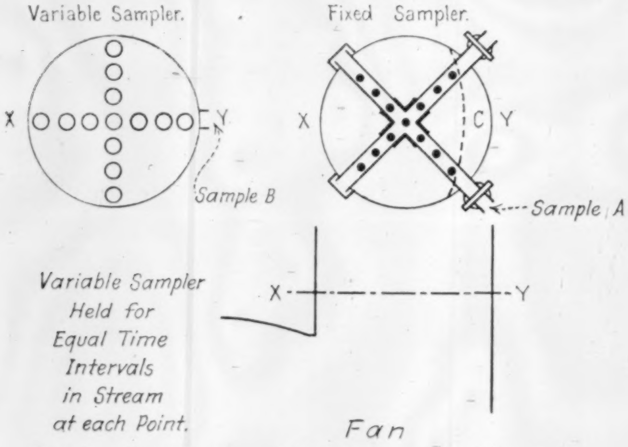
Variable Sampler.
Fixed Sampler.
X
Y
X
C
Y
Sample B
Sample A
Variable Sampler Held for Equal Time Intervals in Stream at each Point.
X
Y
Fan

FIG. 1.—Cross-Section of Pipe Showing Sampling Points.

been taken in that way. All of it passed the No. 200 sieve, whereas the scintillating sparks in the flame would indicate a fineness of only 75 to 80 per cent through that sieve. The reason for this difference is obvious. In practice, mills of several tons per hour capacity will fill a large bag long before steady and uniform conditions of operation are established. Yet in this method, it is customary to cease mill operation, place the bag in position and take the sample. To make such a method accurate requires that the sample be taken during an interval of normal operation and that this balance be not disturbed by the change to the sampling. It is practically impossible to do this when a bag is used, as the frictional resistance in the line is changed, the fan works at a higher pressure and delivers a lesser volume of air, and the coarser particles which are normally delivered to the furnace are not lifted from the mill.

During some recent studies on the fineness of pulverized coal, the author was privileged to compare an interesting sampler built into a pipe with one of his own choosing. The sampling equipment specified in the Society's Tentative Method of Test for Fineness of Powdered Coal (D 197 – 26 T)[1] will be used as a basis of reference in this case and the deviation of the other from it will be noted. A study of Fig. 1 will show the points of difference. Samples taken within a short time of each other gave the following test results:

SIEVE	SAMPLE A (PERMANENT SAMPLER), CUMULATIVE PER CENT	SAMPLE B (A.S.T.M. TYPE SAMPLER), CUMULATIVE PER CENT
Retained on No. 35	0.45	0.37
" " No. 42	0.65	0.77
" " No. 48	1.20	1.95
" " No. 60	2.15	3.16
" " No. 65	3.05	5.42
" " No. 80	4.40	8.72
" " No. 100	7.76	15.73
" " No. 115	10.59	20.74
" " No. 150	16.45	32.26
" " No. 170	22.39	40.22
" " No. 200	26.14	43.36

The former or permanent sampler errs in several particulars. Its outermost openings do not include the band in which much of the coal and most of the coarse particles travel. In its location with respect to the fan, it fails to include the area marked *C* (Fig. 1), where the centrifugal action of the fan tends to throw the coarser particles. It may not even represent the zones it covers. Some of the holes may be clogged and not be blown out prior to sampling, for once the air used to clear the sampler relieves itself through a few of the openings, it can do no further cleaning. Even the holes that are open may function but little, if at all, due to velocity effects in the pipe. The speed of the air stream varies across the pipe and the pressure due to this velocity is different at the several openings. They will thus have an uneven distribution of air. In fact, the pressure in the region of high velocity may so predominate in the sampling tube that the other openings serve as outlets to relieve it rather than as openings for taking the sample.

PRINCIPLES OF SAMPLING

The sampling equipment just discussed shows variations which are due to segregation effects in the pipe and to pressure and velocity effects in the air stream. Some studies have been made to show the magnitude of the former, but there is no adequate data for a study of the latter.

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part I, p. 898 (1926); also 1927 Book of A.S.T.M. Tentative Standards, p. 522.

Segregation Effects:

The Performance Test on the Riley Atrita[1] shows very markedly the extent of the segregation of coarse and fine coal due to the centrifugal action of the fan and the turns in the pipe.

This may be further supplemented by tests taken on one side of a divided pipe, in which the changes of direction in the pipe and the action of the dividers caused segregation.

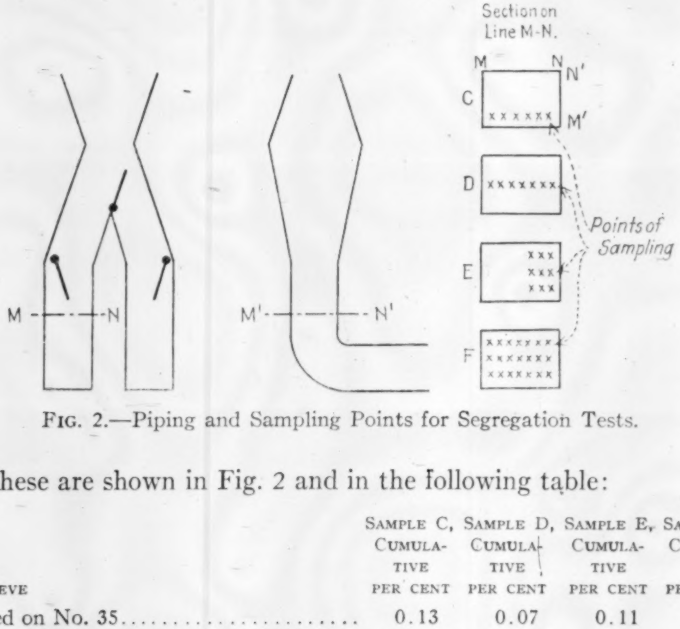

Fig. 2.—Piping and Sampling Points for Segregation Tests.

These are shown in Fig. 2 and in the following table:

Sieve	Sample C, Cumulative per cent	Sample D, Cumulative per cent	Sample E, Cumulative per cent	Sample F, Cumulative per cent
Retained on No. 35	0.13	0.07	0.11	0.05
" " No. 42	0.33	0.24	0.30	0.14
" " No. 48	1.01	0.76	0.97	0.44
" " No. 60	1.95	1.38	2.22	0.98
" " No. 65	4.06	2.65	4.77	1.93
" " No. 80	6.53	4.62	8.06	3.78
" " No. 100	11.13	8.44	14.43	7.63
" " No. 115	14.39	10.93	18.73	10.59
" " No. 150	20.68	15.44	27.00	16.33
" " No. 170	25.13	19.16	32.61	20.48
" " No. 200	28.42	21.79	35.39	23.26

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[1] "Pulverized Fuel," *Bulletin*, Nat. Electric Light Assn., pp. 112–114 (1927).

It is evident that the coarser particles travel along the end and the side of the pipe represented by Samples *C* and *E*, while the complete traverse shows a finer sample. These were all taken through a 1-in. pipe to which a vacuum cleaner bag was attached.

Pressure, Velocity and Resistance Effects:

Besides segregation, there is another more complex sampling problem, that resulting from pressure and velocity effects in the pipe. It is necessary, when diverting a portion of the air stream for sampling, to remove the air at the same rate as it had been moving. Otherwise more or less than the correct amount of air and coal will pass into the sampling tube in a given time. This affects the fineness of the sample, since the fine particles project themselves into the sampling tube irrespective of the diversion or inclusion of the air and fines.

The primary factors which affect this distribution are:

1. The static pressure in the pipe at the plane of sampling. In any installation, this pressure is determined by the type of burner, the length and size of pipe, and the flow of air. Qualitative inspection of equipment when sampling shows that the rates of air flow differ considerably, in a manner which is not likely to be proportional to the rate of flow in the pipe.

2. The different pressures due to velocity at each sampling point. These will affect the rate of air flow into the sampler and thus the relative amounts of coarse and fine coal in the sample. The values vary from point to point in the cross-section of the pipe. Some measurements made with a pitot tube show values ranging from 0.50 to 1.74 in. They do not vary as might be expected for either viscous or turbulent flow of air, but rather with respect to the turns or bends in the pipe. Were it not for the frictional resistance of the sampler circuit, a sampling scheme utilizing velocity pressure would offer a good solution to this problem.

3. The resistances to fluid flow of the sampler circuit and the conveyor pipe. There is lack of specific knowledge as to the magnitude of this. As a consequence, many types of equipment have been used to eliminate the effect of resistance. There is as yet no automatically compensating scheme to do this. The pipe and vacuum cleaner bag specified in the method for unit systems described in the Tentative Method of Test for Fineness of Powdered Coal (D 197 - 26 T) of the American Society for Testing Materials[1] utilizes the internal pressures in the pipe and is simple of construction. Jet or

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[1] *Proceedings*, Am. Soc. Testing Mats., Vol. 26, Part I, p. 898 (1926); also 1927 Book of A.S.T.M. Tentative Standards, p. 522.

mechanical blowers have been used to overcome the frictional resistance. While this is necessary in sampling under vacuum, its cautious use is required at all times. In comparison with the standard sampler, such equipment will give a finer powder.

Resistance is affected by the dust collectors which are of two types, filter and cyclone. The former is commonly a vacuum cleaner bag. This provides sufficient surface to keep down the resistance to air flow. The latter gives very low air resistances, but, lacking a filtering surface, it permits some loss of dust.

SAMPLING PROCEDURE

As the problem of sampling coal from air streams now stands, several of the difficulties noted here may be eliminated. Errors due to segregation, which have been so largely the cause of inaccurate results, may be tested for and guarded against. Samples should be collected at several spots at the desired cross-section of the pipe and their fineness and total weights observed. Such differences as exist will indicate the magnitude of the segregation. The section where the greatest uniformity exists is obviously the best place for sampling. Thus it is well not to sample near bends or dampers nor in long horizontal sections of pipe, particularly when the velocity is low.

The sample which is taken should be a composite from several points, so selected as to represent the whole area. Mechanical difficulties ordinarily prevent a complete traverse. It is customary to withdraw samples for a fixed time at even intervals along one line. If near the fan off-take, that line should be in the plane of the fan to include areas carrying the largest proportion of solid. It sometimes happens that the direction of the fan intake throws the maximum content of coal slightly to one side of this. In a circular pipe, a second traverse, at right angles to this, compensates for the extra width at the center and prevents loss of fine or coarse material which is carried along the sides of the pipe.

The magnitude of these effects may be determined experimentally by tests on a storage system installation. There the large cyclone separators, used with air-swept mills, collect all of the coal under conditions of steady operation. Only a slight amount of dust is lost at the vents. The basis for conclusive tests of sampling from air streams lies in a comparison of samples so taken with those taken simultaneously from the cyclone product.

CONCLUSIONS

In the sampling of pulverized coal from air streams, wide variations in results are encountered:

(*a*) Through failure to remove coal from a representative cross-section, there being segregation of coarse and fine coal in different sections of the area of the pipe.

(*b*) Through failure to draw into the sampling tube material representative of the spot under test, there being diversion or inclusion of air and fines in accordance with pressure, velocity, and resistance effects in the coal pipe and in the sampling equipment.

With the present types of sampling procedures, segregation errors can best be avoided:

(*a*) By using a sampler with a single opening, like that specified in the present recommended procedure of the Society.

(*b*) By withdrawing portions of the sample from sufficiently representative points in the cross-section of the pipe, and,

(*c*) By selecting the section of most uniform flow in the pipe, away from bends, fan action, and great horizontal lengths of pipe.

Pressure, velocity and resistance effects are of unknown extent. Tests made in the same manner upon the same installations are comparable. To correlate data on different plants, the best method at present is to use one type of sampler and to make record of the conditions of its use.

METHODS OF MEASURING THE THERMAL CONDUCTIVITY OF INSULATING AND REFRACTORY MATERIALS

By M. L. Hartmann,[1] O. B. Westmont[2] and C. E. Weinland[3]

Synopsis

A series of thermal conductivity measurements was carried out to investigate methods that might be used as standard tests. The apparatus was of the guarded hot plate type and was used for making conductivity determinations at mean temperatures up to approximately 1800° F.

Methods are described for the use of the same apparatus to determine the thermal conductivity of (1) low-temperature insulators, (2) medium to high-temperature insulators, and (3) refractory materials. Experimental results of the conductivity measurements on materials included in these three classes are given. Thermal conductivity curves are given to show the relationship of conductivity to mean temperatures and to density.

The precision of the measurements after applying all corrections is estimated to be about 2.5 per cent.

The apparatus and the methods described appear to be of sufficient reliability to be given consideration in the establishment of specifications and standards.

The thermal conductivity of any uniform material is a specific property, and is defined as the time rate of heat flow (after a steady state has been established) per unit area per unit temperature gradient. It is further defined as the time rate of heat flow in one direction with the lines of heat flux perpendicular to the plane of an area. This absolute conductivity, as it might be called, is a function of temperature but it is not dependent upon the area, shape, surface resistances, differences in pressure, etc. These factors affect the total quantity of heat transmitted.

The relationship of conductivity to temperature is influenced by the size and shape of pores in the material, since the heat flow is by radiation and convection within the pores as well as by conduction through the solid particles. The heat flow by radiation and convection increases rapidly with temperature so that the conductivity-temperature coefficient of an insulator with large pores is greater than that of the same type of insulator of the same bulk density but with very fine pores.

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[1] Technical Director, The Celite Co., Lompoc, Calif.

[2] Research Chemist, The Celite Co., Lompoc, Calif.

[3] Assistant Physicist, The Celite Co., Lompoc, Calif.

The lack of agreement in the literature of the thermal conductivity data for similar materials is partly due to the misunderstanding of the meaning of thermal conductivity and overall heat transmission. In some cases the conductivity of the material is a small factor in the overall transfer of heat, as for example, steam pipes with comparatively high surface and contact resistances. In all building and furnace insulation the internal conductivity of the materials is the most important factor affecting heat transfer.

It is, therefore, essential that the true conductivity of these insulating materials be measured accurately in order that (1) comparisons can be made of their relative insulating values, and (2) the temperatures in walls and the heat losses can be calculated. It has been stated that the variations of "conditions" of practice apparently have a greater effect on the heat transfer than the errors or discrepancies in the conductivity coefficients. This emphasizes the importance of having some accurate basis on which to determine these variations and to measure such factors as surface and joint resistances, diffusivity, effects of gas pressure, etc.

In order to secure accurate and comparable thermal conductivities of insulating and refractory materials it is essential that some standard test method be adopted and the results of tests on the same samples be checked by various investigators. It is also desirable that the same terminology and methods of expressing results be used so there will be an agreement in all thermal calculations.

RECENT METHODS OF MEASURING THERMAL CONDUCTIVITY

The guarded calorimeter and hot plate methods have been used extensively for measuring the thermal conductivity of insulating and refractory materials. The most practical calorimeter method was employed by Dudley.[1] This method with several refinements was used by the authors[2] in making high-temperature conductivity tests on typical furnace walls. The apparatus for measuring the heat flow consisted of an 8-in. square calorimeter and two 2-in. guard rings, all of which were provided with means for double circulation of water. The temperature gradients through the wall were measured simultaneously at three points directly under the calorimeter by means of thermocouples inserted in the wall to any measured position.

This type of apparatus was quite satisfactory for high-temperature tests on refractory materials. The precision of the measurement would probably be increased by providing several fixed protected thermocouples in the wall and placing the lead wires perpendicular to the direction of heat flow. This method requires large and extensive equipment and is not so convenient nor so well adapted to laboratory conditions as the hot plate method for making standard tests on the conductivity of various materials.

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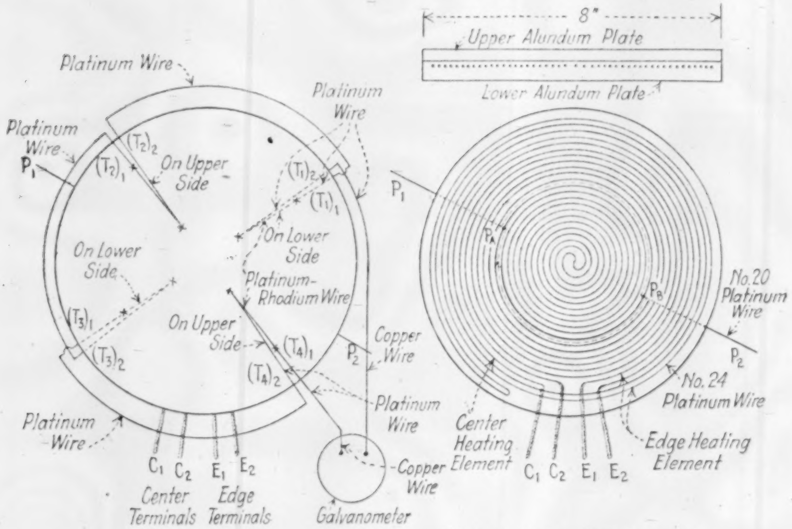
[1] B. Dudley, Jr., *Transactions*, Am. Electrochemical Soc., Vol. 27, p. 285 (1915).

[2] M. L. Hartmann and O. B. Westmont, *Transactions*, Am. Electrochemical Soc., Vol. 46, p. 485 (1924).

In the plate method the heat flow is measured by the power supplied to a central heater plate that is provided with a guard ring to prevent edge losses. Comparatively thin specimens are used and the temperature gradient measured by surface thermocouples. In this investigation several modifications in the usual plate method were made so that the same apparatus could be used for measuring the true conductivity of (1) low-temperature insulators, (2) medium to high-temperature insulators, and (3) refractory materials.


Platinum Wire
Platinum Wire
P_1
$(T_2)_2$
$(T_2)_1$
On Upper Side
Platinum Wire
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On Lower Side
On Lower Side
Platinum-Rhodium Wire
$(T_3)_1$
$(T_3)_2$
On Upper Side
$(T_4)_1$
$(T_4)_2$
P_2
Copper Wire
Platinum Wire
Platinum Wire
C_1 C_2 E_1 E_2
Center Terminals
Edge Terminals
Copper Wire
Galvanometer
8"
Upper Alundum Plate
Lower Alundum Plate
P_1
P_A
P_B
No. 20 Platinum Wire
P_2
No. 24 Platinum Wire
Center Heating Element
C_1 C_2 E_1 E_2
Edge Heating Element

Plan of Assembled Heater Plate Showing Location of Differential Thermocouples.

Plan of Lower Alundum Plate with Platinum Heating Elements. (Power Measured between P_A and P_B).

FIG. 1.—Thermal Conductivity Heater Plate.

DESCRIPTION OF APPARATUS

The apparatus used was originally designed at the Bureau of Standards by M. S. Van Dusen[1] and is somewhat similar to the apparatus employed for measuring the conductivity of low-temperature insulators. This apparatus is also described by Calvert and Caldwell,[1] but since the publication of their description it has been modified as shown in Figs. 1 and 2.

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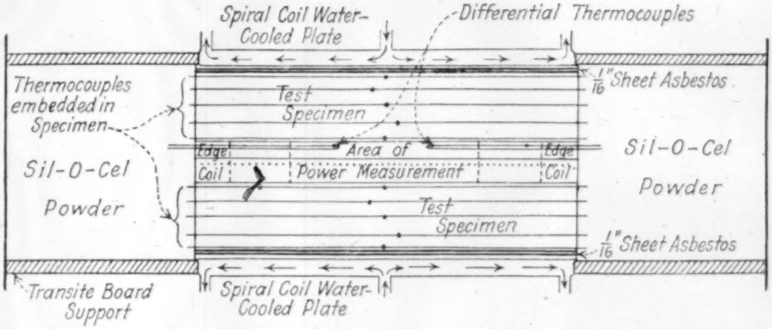
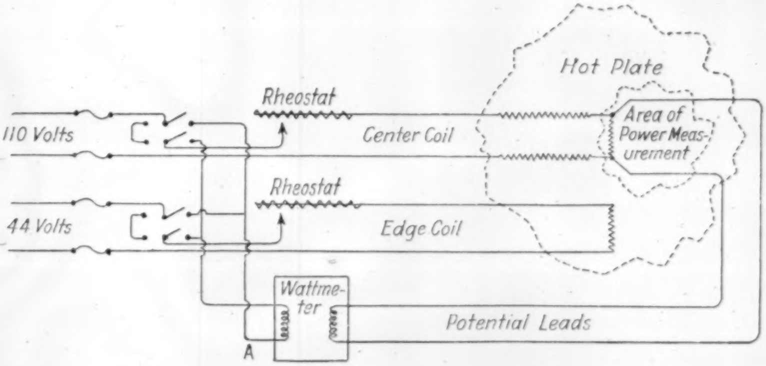
[1] M. S. Van Dusen, *Journal*, Am. Soc. Heating and Ventilating Engrs., Vol. **26**, **p. 625** (**1920**).

The heating plate, Fig. 1, is constructed of two 8-in. disks that were molded from a stiff mud mixture of 70 per cent fine and 30 per cent coarse Alundum cement. The lower plate is 0.5 in. thick and contains double spiral grooves $\frac{3}{32}$ in. deep spaced $\frac{1}{8}$ to $\frac{3}{16}$ in. apart for the wire heating elements. The upper plate is $\frac{3}{8}$ in. thick so that the outside surfaces are equi-distant from the heater elements when the two plates are cemented together.


Spiral Coil Water-Cooled Plate
Differential Thermocouples
Thermocouples embedded in Specimen
Test Specimen
$\frac{1}{16}$" Sheet Asbestos
Edge Coil
Area of Power Measurement
Edge Coil
Sil-O-Cel Powder
Sil-O-Cel Powder
Test Specimen
$\frac{1}{16}$" Sheet Asbestos
Transite Board Support
Spiral Coil Water-Cooled Plate

Section of Thermal Conductivity Apparatus.


Hot Plate
Rheostat
110 Volts
Center Coil
Area of Power Measurement
Rheostat
44 Volts
Edge Coil
Wattmeter
A
Potential Leads

Schematic Wiring Diagram of Power Circuits.

FIG. 2.—Thermal Conductivity Apparatus.

The heating elements consist of No. 24 B. & S. gage platinum wire arranged in the form of non-inductive windings. The center element extends over 6.5 in. of the plate but the area in which the power is measured is approximately 4 in. in diameter as shown by the section between the potential lead connections P_A and P_B in Fig. 1. The edge element is wound concentric with the center coil and covers the outer $\frac{3}{4}$ in. of the plate.

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[1] R. Calvert and L. Caldwell, *Industrial and Engineering Chemistry*, Vol. 16, No. 5, p. 483 (1924).

The differential thermocouples are made of No. 30 B. & S. gage platinum and platinum-rhodium alloy wire with $1\frac{1}{2}$ in. between junctions. Two couples are embedded in the surface of each plate with one junction $1\frac{1}{4}$ in. from the center and the other junction the same distance from the edge. The four couples are connected in series to a sensitive reflecting galvanometer to detect small differences in temperature over the surfaces of the heater plate.

The thermocouples for measuring the temperatures in the specimens at points near the heater plate are No. 30 B. & S. gage platinum - platinum-rhodium elements while those near the cooling plates are the same size Chromel-Alumel wire. The interior temperatures of the test specimens are measured with thermocouples drawn through $\frac{3}{64}$ to $\frac{1}{16}$-in. holes drilled perpendicular to the direction of heat flow. All the couples are connected to a cold junction box and the e.m.f.'s measured by a potentiometer that is read directly to 0.02 millivolt.

The assembly of the apparatus with test specimens placed between the heater and the spiral coil water-cooled plates is shown in Fig. 2. The $\frac{1}{16}$-in. sheet of asbestos is inserted between the specimens and the cooling plates to insure good contact. The apparatus is connected to 60-cycle, 44 and 110-volt power lines and adjustments made with Nichrome and carbon plate rheostats.

CALIBRATION OF APPARATUS

The differential thermocouples used for measuring the gradients in the low-temperature conductivity tests were made of Chromel-Alumel wire and calibrated against standard thermometers to read temperature differences to ±0.5° F.

The thermocouples for the other tests were calibrated by comparison with a standard rare metal couple. This couple was previously calibrated against the known freezing points of zinc, aluminum, and copper, from the Bureau of Standards. The base metal couples were not used at temperatures above 1000° F. since it was found that their calibrations changed slightly after prolonged heating at higher temperatures.

The power supplied to the center heater was measured with a Weston Model 310 wattmeter which had been recently standardized. This meter was not connected continuously in the line during the tests since the current was often near the maximum capacity of the instrument. When the meter was connected in the circuit the added resistance reduced the indicated power. This correction was measured with two wattmeters and applied to all indicated readings.

In the conductivity tests at high temperatures a potential drop was indicated between the thermocouples, differential couples, and the ground. There was also a voltage drop between the center heater element and the ground when this element was disconnected and the edge heater was in circuit. This effect was appreciable above about 1500° F. which indicated that the electrical conductivity of Alundum increased sufficiently at high temperatures to cause some current leakage. This leakage would affect the indicated power in the center area.

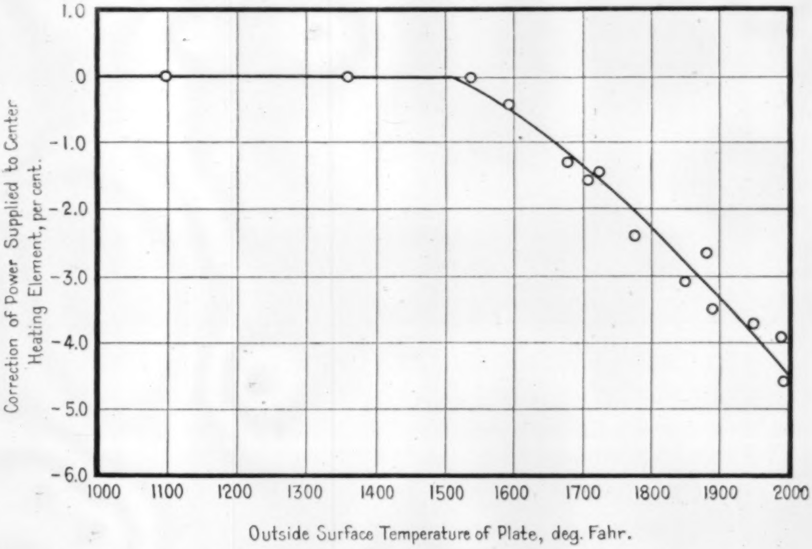

FIG. 3.—Power Loss in Alundum Heating Plate at High Temperatures.

The magnitude of this power loss was measured with two wattmeters, one connected as usual and the other connected to two auxiliary leads that were brought out perpendicularly through one face of the plate. These leads consisted of two potential connections from the potential junctions and two current connections from a break in the center coil just outside the area of power measurement. The plate was assembled between insulators as in a regular conductivity test and with increasing temperatures the power loss measured as shown in Fig. 3. This loss is 4.5 per cent at 2000° F. which shows that Alundum is not a satisfactory electrical insulator for precise measurements at high temperatures. In using Alundum in such measurements it is therefore, necessary to determine the characteristics of the particular apparatus by a blank test and thus eliminate a serious error.

PREPARATION OF SPECIMENS

The samples for thermal conductivity tests were oven dried thoroughly at 250° F. and the density of each determined by the sand displacement method.[1] The samples with not over 0.5 lb. per cu. ft. variation were then selected and the average porosity measured by air expansion.[1]

Two conductivity specimens were cut from these samples in the form of disks 8 in. in diameter and about 1.4 in. in thickness with the surfaces plane and parallel to within 0.002 in. The joints between brick were in all cases at least 2 in. from the center of the specimen so that the joint resistance would not affect the temperatures recorded by the thermocouples. The small holes for the couple wires were drilled through the center section of each disk. Two of these holes were located about $\frac{3}{8}$ in. from the axis of the specimen. In the case of soft materials only one or two holes were provided. After the thermocouples were drawn through the specimen with their junctions near the center, the holes were filled with powdered material of the test sample. The couples on the outside surfaces were cemented in shallow grooves with a mixture of the powdered material and bentonite. The specimens were then covered with powder to fill the joints and surface pores, and assembled in the apparatus.

In two tests each specimen consisted of thin plates carefully ground together. One test was with four thin plates of calcined Sil-O-Cel and the other test with two plates cut from fire-clay tile. Thermocouples were embedded in the surfaces of each plate in order to measure the joint resistances as well as the temperature gradients.

In preparing C-3 concrete specimens the thermocouples were arranged at various depths as the mixture was uniformly tamped in the molds. The concrete was then cured in moist air 28 days, oven dried at 250° F. and fired slowly to 800° F. to dehydrate the material.

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[1] M. L. Hartmann, O. B. Westmont, and S. F. Morgan, *Journal*, Am. Ceramic Soc., Vol. 9, No. 5, p. 298 (1926).

METHODS OF TEST

Low-Temperature Insulators:

This class of insulators is arbitrarily selected to include all organic and hydrated materials that cannot be heated to temperatures above about 400° F. without changing their composition.

In the low temperature tests no interior couples were used but the temperature gradient through the specimens was measured with Chromel-Alumel differential couples made with four junctions in series. One ordinary base metal couple was attached to the hot surface of each specimen in order to record that temperature.

The power was measured with a specially designed Weston wattmeter which was connected to give a full scale deflection of 7.5 watts and read directly to 0.05 watt.

With this type of insulation the temperature was increased very slowly until the hot surface temperatures were near the maximum that the material would withstand. The power in both heater coils was then adjusted until the surface temperatures of the plate were uniform, as indicated by the galvanometer. When the hot surface temperatures were practically constant, all the temperatures, power, current, and galvanometer readings were taken every 30 minutes. The power input and galvanometer deflection were read every five minutes during the time of equilibrium so that adjustments could be made for slight fluctuations before they affected the heat flow.

In these tests equilibrium conditions were indicated when the temperature difference did not vary more than 0.3° F. for at least $1\frac{1}{2}$ hours; the power was constant for at least 2 hours, and the galvanometer indicated a temperature balance between the center and edge of the hot plate to within 0.5° F. With good insulators it usually required about four days to reach this thermal equilibrium.

Medium to High-Temperature Insulators:

This class includes all the insulating materials that are used at temperatures ranging from about 400 to 2500° F. and have a thermal conductivity of less than 3.0 B. t. u. per sq. ft. per hour per inch per 1° F.

In the tests on rigid insulators the apparatus was arranged as shown in Fig. 2, but in the case of powders and aggregates, calcined Sil-O-Cel rings were provided to hold the material in place. These rings were 1.4 in. in thickness and 7.9 in. in inside diameter. The insulation was uniformly packed to the desired density in the rings and two or three thermocouples stretched across the center at different depths to measure the temperature gradients.

In making the test the power was adjusted so that the rate of heating was about 100° F. per hour until equilibrium was reached. This was indicated when all the thermocouples showed no more than 4° F. change in temperature for at least 1½ hours and the temperature balance between the center and edge of the hot plate was within 1° F. The time required to obtain such conditions usually varied from 15 to 24 hours. After a satisfactory equilibrium was obtained the apparatus was heated or cooled as desired at the rate of 100° F. per hour to the next equilibrium point. The tests were made with hot surface temperatures up to the maximum which the material would withstand without shrinking or warping.

The specimens were carefully removed after completion of the tests and the depth of the surface couple junctions measured with vernier calipers. A section of the specimen was then ground away and the depth of the thermocouple junction measured accurately with a depth gage. From these measurements and the recorded temperatures the average gradient through the two test specimens was plotted for each equilibrium as shown in Figs. 4 to 17, inclusive.

Refractory Materials:

The same method for measuring the conductivity of the common refractories, namely, fire clay, silica, fused alumina, silicon carbide, etc., can be used for testing other highly conductive materials such as red brick and building tile, etc. The test as described cannot be used for making high-temperature measurements on account of the excessive conduction of heat to the cooling plates. In order to secure higher mean temperatures with low power the resistance to heat flow was increased by inserting ⅜-in. calcined Sil-O-Cel insulating disks between the specimens and the water-cooled plates. These plates were made with carefully ground and parallel surfaces.

By the use of this insulation the same effect was produced as with the auxiliary heating plates employed by Heilman,[1] but less equipment was required and the control greatly simplified.

The specimens were not provided with interior thermocouples since most of the refractory materials are difficult to drill. In addition, the temperature gradient was so small in comparison to that in insulating materials that additional measurements were not required. The test was conducted as described above and the mean conductivity determined for at least four equilibrium points.

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[1] R. H. Heilman, *Transactions*, Am. Inst. Chemical Engrs., Vol 18, p. 283 (1926).

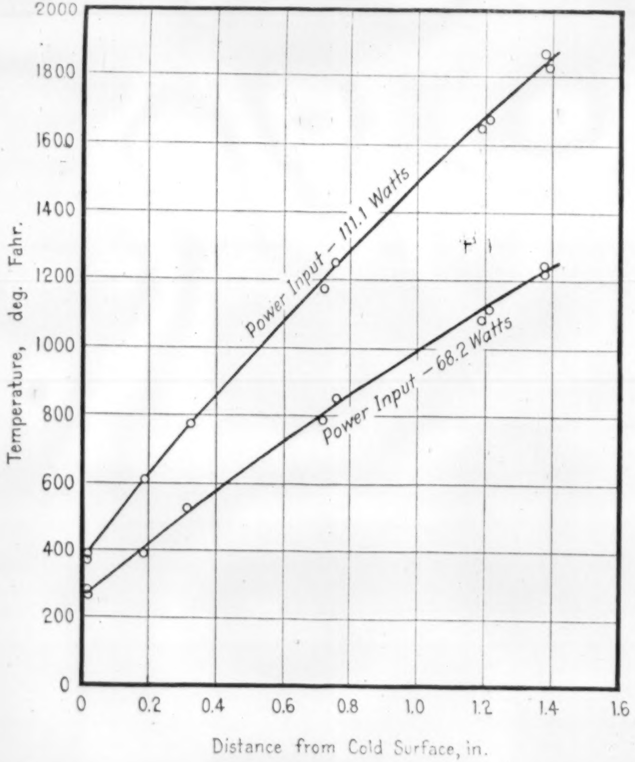

FIG. 4.—Temperature Gradients Through Sil-O-Cel C-22 Brick (Specimen A).

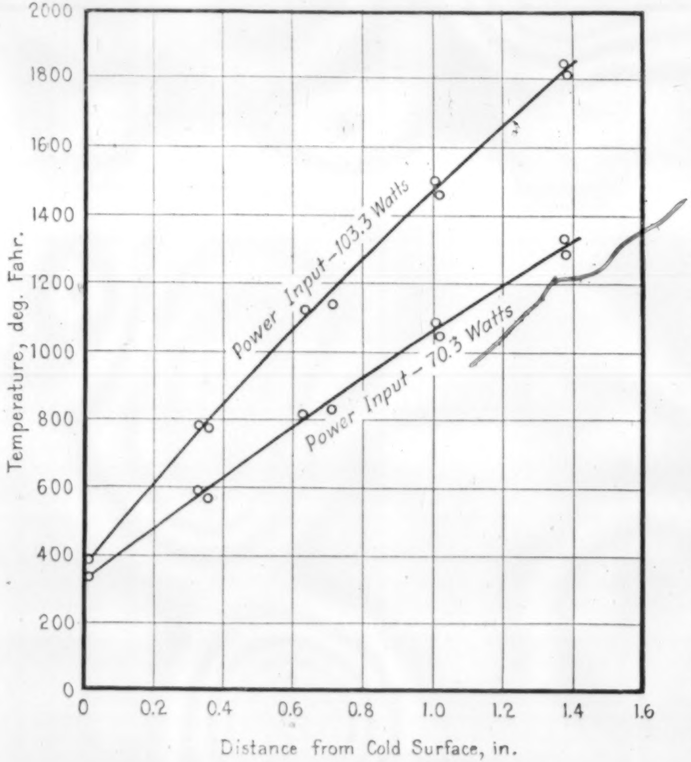

FIG. 5.—Temperature Gradients Through Sil-O-Cel C-22 Brick (Specimen B).

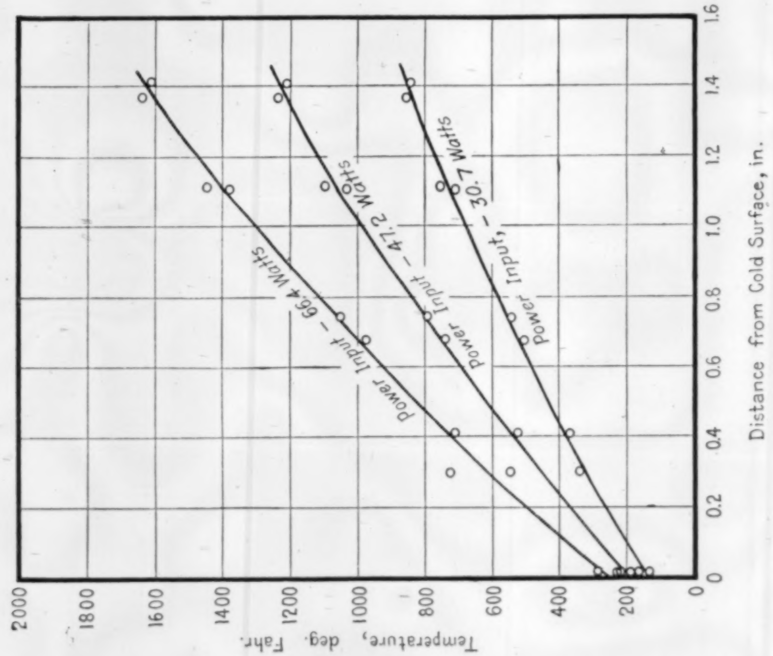

FIG. 6.—Temperature Gradients Through Special Sil-O-Cel.

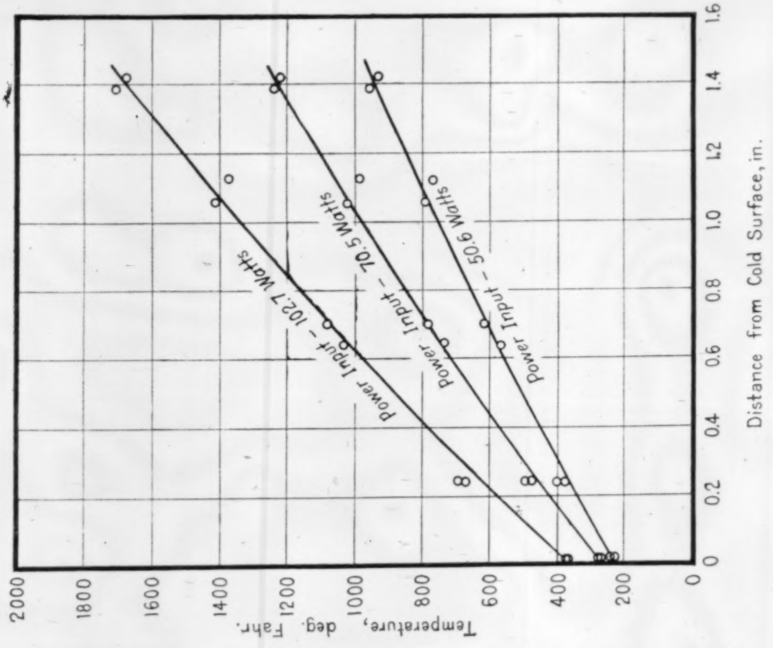

FIG. 7.—Temperature Gradients Through Sil-O-Cel Super Brick (Specimen A).

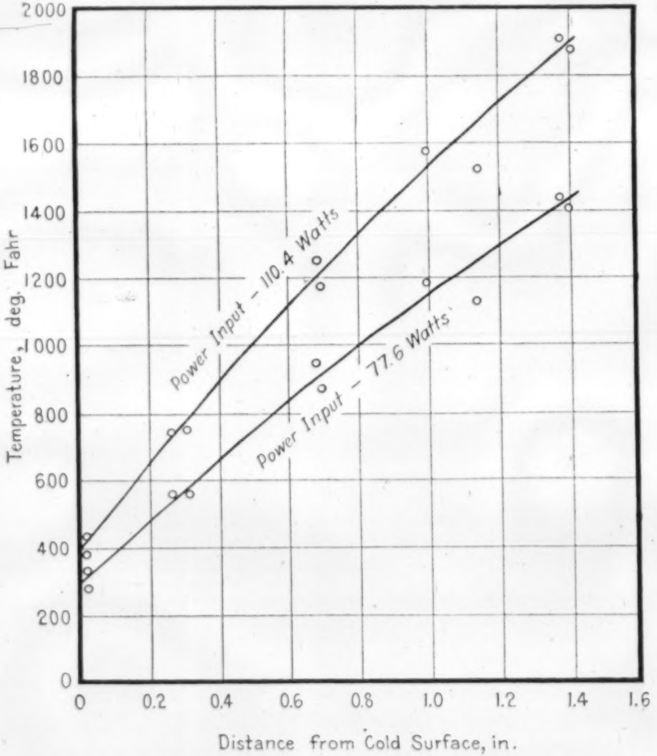

FIG. 8.—Temperature Gradients Through Sil-O-Cel Super Brick (Specimen B).

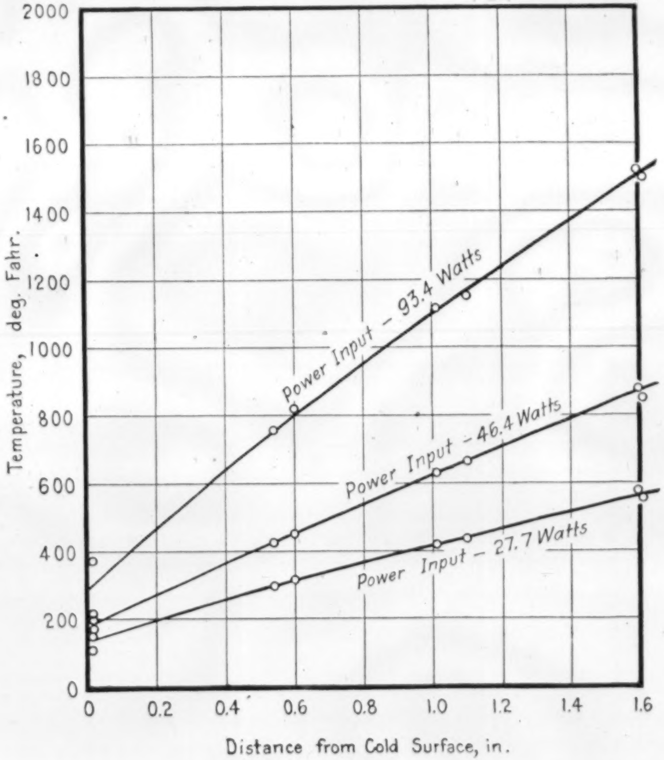

FIG. 9.—Temperature Gradients Through Sil-O-Cel C-3 Concrete.

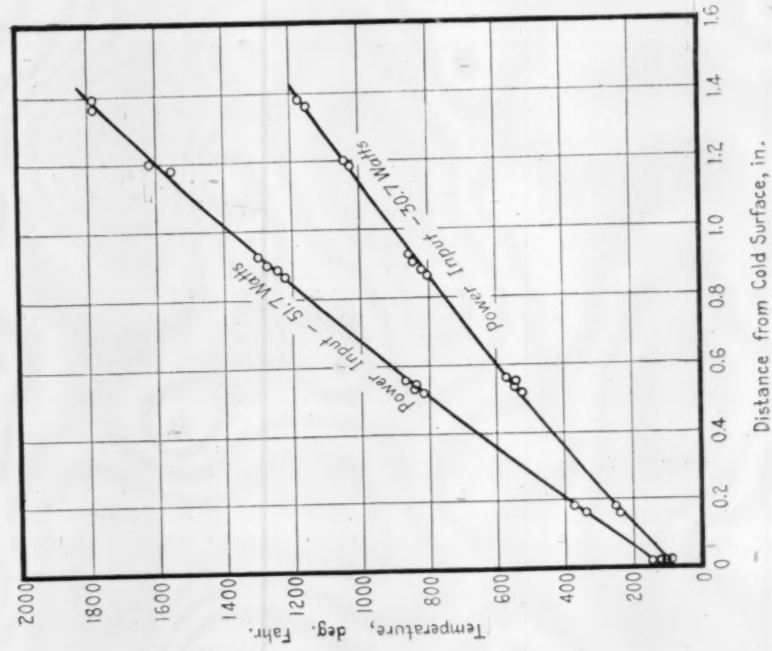

FIG. 10.—Temperature Gradients Through Calcined Sil-O-Cel Brick (Specimen A).

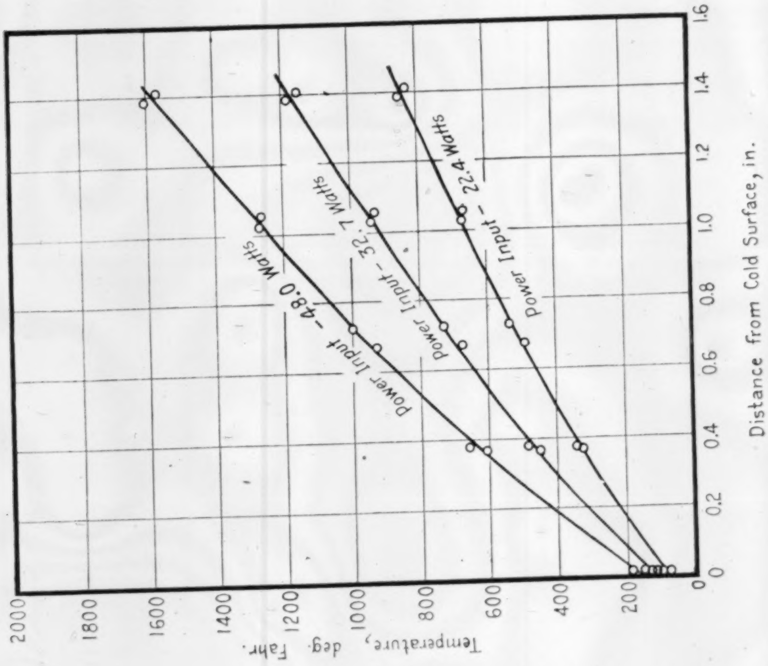

FIG. 11.—Temperature Gradients Through Calcined Sil-O-Cel Brick (Specimen B).

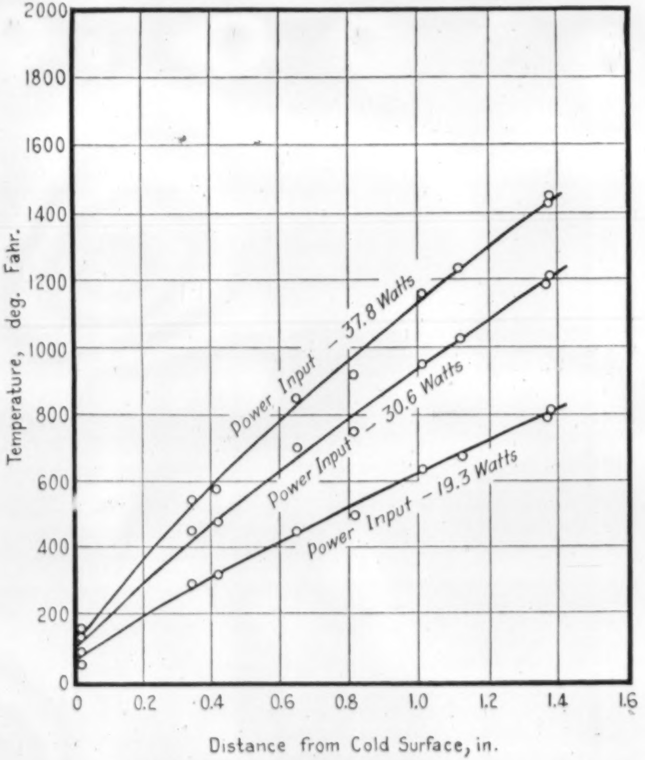

FIG. 12.—Temperature Gradients Through Sil-O-Cel Natural Brick (Specimen A.)

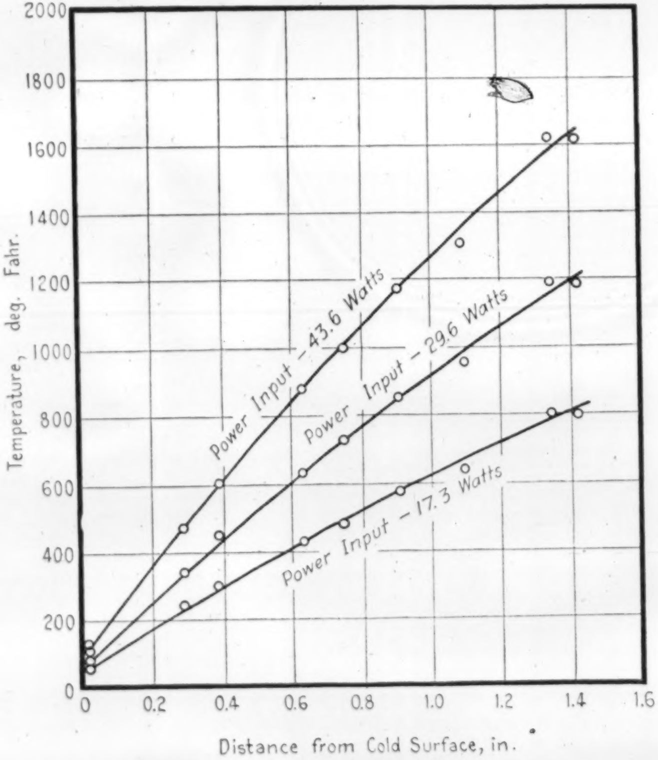

FIG. 13.—Temperature Gradients Through Sil-O-Cel Natural Brick (Specimen B).

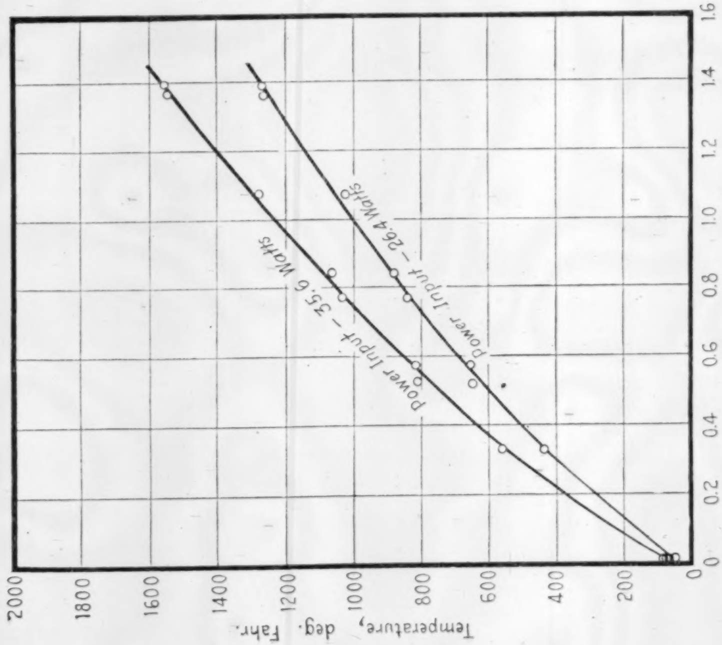

FIG. 14.—Temperature Gradients Through Sil-O-Cel Powder (Specimen A.)

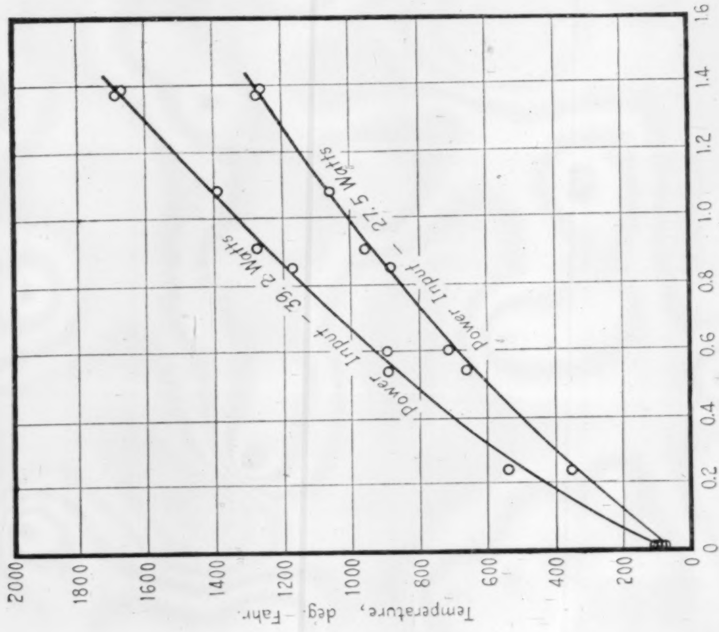

FIG. 15.—Temperature Gradients Through Sil-O-Cel Powder (Specimen B).

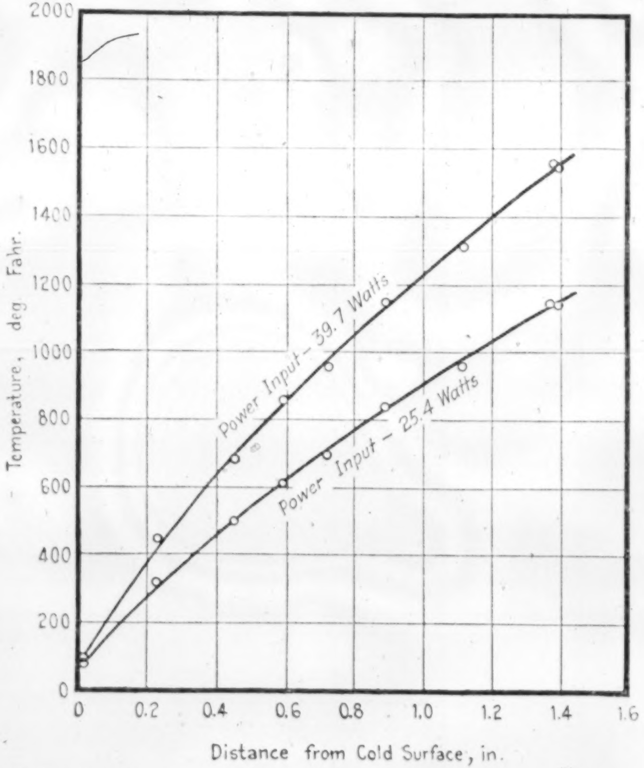

FIG. 16.—Temperature Gradients Through Sil-O-Cel Coarse Grade (Specimen A).

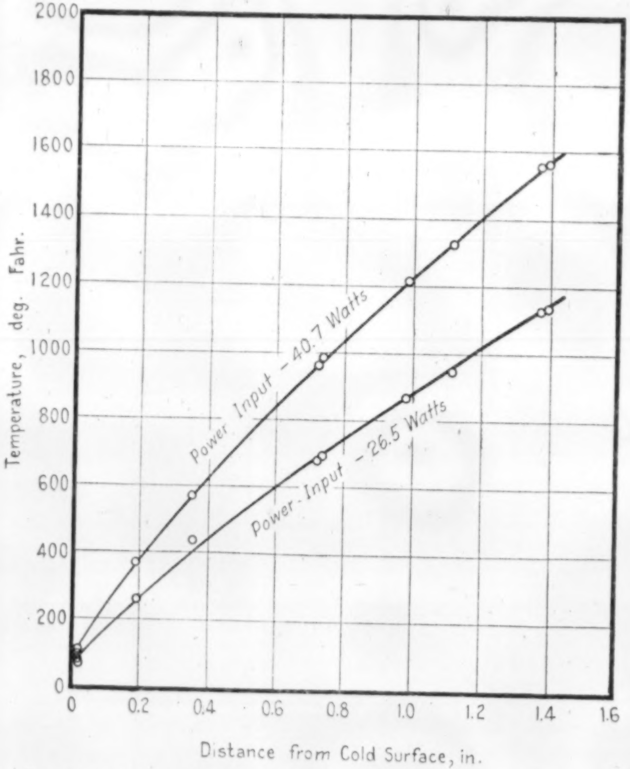

FIG. 17.—Temperature Gradients Through Sil-O-Cel Coarse Grade (Specimen B).

TABLE I.—THERMAL CONDUCTIVITY OF INSULATING BRICK AND CONCRETE.

Dimension x_1, in.	Dimension x_2, in.	Distance, $x_2 - x_1$, in.	Temperature T_1, deg. Fahr.	Temperature T_2, deg. Fahr.	Temperature Difference $T_2 - T_1$, deg. Fahr.	Mean Temperature, $\frac{T_1 + T_2}{2}$, deg. Fahr.	Watts Indicated	Watts Corrected	k, B.t.u. per sq. ft. per hour per inch per 1° F.
TEST No. 1, HYDRATED INSULATING MATERIAL, DENSITY 26.0 LB. PER CU. FT.									
0.00	1.38	1.38	61.8	147.5	85.7	105	1.300	1.304	0.397
TEST No. 2, SIL-O-CEL C-22 BRICK, DENSITY 40.0 LB. PER CU. FT., POROSITY 75.0 PER CENT (SPECIMEN A).									
0.01	0.70	0.69	267	784	517	525	66.0	68.2	1.72
0.70	1.38	0.68	784	1220	436	1002	66.0	68.2	2.00
0.01	1.38	1.37	267	1220	953	743	66.0	68.2	1.85
0.01	0.70	0.69	393	1175	782	784	110.0	111.1	1.85
0.70	1.38	0.68	1175	1839	664	1507	110.0	111.1	2.15
0.01	1.38	1.37	393	1839	1446	1116	110.0	111.1	1.99
TEST No. 3, SIL-O-CEL C-22 BRICK, DENSITY 36.0 LB. PER CU. FT., POROSITY 75.7 PER CENT (SPECIMEN B).									
0.01	0.70	0.69	290	855	565	572	68.0	70.3	1.62
0.70	1.35	0.65	855	1295	440	1075	68.0	70.3	1.96
0.01	1.35	1.34	290	1295	1005	792	68.0	70.3	1.77
0.01	0.70	0.69	382	1172	790	777	102.0	103.3	1.70
0.70	1.35	0.65	1172	1800	628	1486	102.0	103.3	2.02
0.01	1.35	1.34	382	1800	1418	1091	102.0	103.3	1.85
TEST No. 4, SPECIAL SIL-O-CEL BRICK, DENSITY 33.8 LB. PER CU. FT., POROSITY 76.7 PER CENT.									
0.01	0.70	0.69	152	522	370	337	30.0	30.7	1.08
0.70	1.35	0.65	522	837	315	679	30.0	30.7	1.20
0.01	1.35	1.34	152	837	685	494	30.0	30.7	1.13
0.01	0.70	0.69	208	758	550	483	46.0	47.2	1.12
0.70	1.35	0.65	758	1204	446	981	46.0	47.2	1.30
0.01	1.35	1.34	208	1204	996	706	46.0	47.2	1.20
0.01	0.70	0.69	259	1008	749	633	65.0	66.4	1.15
0.70	1.35	0.65	1008	1597	589	1302	65.0	66.4	1.38
0.01	1.35	1.34	259	1597	1338	928	65.0	66.4	1.26
TEST No. 5, SIL-O-CEL SUPER BRICK, DENSITY 43.9 LB. PER CU. FT., POROSITY 72.6 PER CENT (SPECIMEN A).									
0.01	0.70	0.69	243	609	366	426	49.0	50.6	1.77
0.70	1.40	0.70	609	944	335	776	49.0	50.6	1.99
0.01	1.40	1.39	243	944	701	593	49.0	50.6	1.89
0.01	0.70	0.69	270	772	502	521	68.0	70.5	1.83
0.70	1.40	0.70	772	1223	451	997	68.0	70.5	2.06
0.01	1.40	1.39	270	1223	953	746	68.0	70.5	1.94
0.01	0.70	0.69	368	1080	712	724	100.0	102.7	1.88
0.70	1.40	0.70	1080	1690	610	1385	100.0	102.7	2.22
0.01	1.40	1.39	368	1690	1322	1026	100.0	102.7	2.03
TEST No. 6, SIL-O-CEL SUPER BRICK, DENSITY 40.7 LB. PER CU. FT., POROSITY 76.7 PER CENT (SPECIMEN B).									
0.01	0.70	0.69	310	930	620	620	75.0	77.6	1.63
0.70	1.35	0.65	930	1400	470	1165	75.0	77.6	2.02
0.01	1.35	1.34	310	1400	1090	855	75.0	77.6	1.80
0.01	0.70	0.69	380	1240	860	810	110.0	110.4	1.67
0.70	1.35	0.65	1240	1875	635	1557	110.0	110.4	2.14
0.01	1.35	1.34	380	1875	1495	1127	110.0	110.4	1.87

TABLE I.—THERMAL CONDUCTIVITY OF INSULATING BRICK AND CONCRETE (*Continued*).

Dimension x_1, in.	Dimension x_2, in.	Distance, $x_2 - x_1$, in.	Temperature T_1, deg. Fahr.	Temperature T_2, deg. Fahr.	Temperature Difference $T_2 - T_1$, deg. Fahr.	Mean Temperature; $\frac{T_1 + T_2}{2}$, deg. Fahr.	Watts Indicated	Watts Corrected	k, B.t.u. per sq. ft. per hour per inch per 1° F.
TEST NO. 7, SIL-O-CEL C-3 CONCRETE (DEHYDRATED AT 800° F.) DENSITY 57.6 LB. PER CU. FT., POROSITY 65.2 PER CENT.									
0.01	0.80	0.79	136	365	229	251	27.0	27.7	1.80
0.80	1.61	0.81	365	570	205	468	27.0	27.7	2.07
0.01	1.61	1.60	136	570	434	353	27.0	27.7	1.93
0.01	0.80	0.79	181	542	361	362	45.0	46.4	1.92
0.80	1.61	0.81	542	869	327	706	45.0	46.4	2.17
0.01	1.61	1.60	181	869	688	525	45.0	46.4	2.04
0.01	0.80	0.79	290	967	677	629	90.0	93.4	2.06
0.80	1.61	0.81	967	1520	553	1244	90.0	93.4	2.58
0.01	1.61	1.60	290	1520	1230	905	90.0	93.4	2.29
TEST NO. 8, CALCINED SIL-O-CEL BRICK, DENSITY 28.1 LB. PER CU. FT., POROSITY 83.9 PER CENT (SPECIMEN A).									
0.01	0.70	0.69	100	680	580	390	30.0	30.7	0.690
0.70	1.37	0.67	680	1162	482	921	30.0	30.7	0.806
0.01	1.37	1.36	100	1162	1062	631	30.0	30.7	0.742
0.01	0.70	0.69	137	1045	908	591	52.0	51.7	0.742
0.70	1.37	0.67	1045	1790	745	1418	52.0	51.7	0.878
0.01	1.37	1.36	137	1790	1653	964	52.0	51.7	0.804
TEST NO. 9, CALCINED SIL-O-CEL BRICK, DENSITY 31.8 LB. PER CU. FT., POROSITY 84.0 PER CENT (SPECIMEN B).									
0.01	0.70	0.69	107	506	399	307	22.0	22.4	0.731
0.70	1.40	0.70	506	846	340	676	22.0	22.4	0.871
0.01	1.40	1.39	107	846	739	478	22.0	22.4	0.795
0.01	0.70	0.69	135	704	569	420	32.0	32.7	0.748
0.70	1.40	0.70	704	1170	466	937	32.0	32.7	0.927
0.01	1.40	1.39	135	1170	1035	653	32.0	32.7	0.828
0.01	0.70	0.69	164	960	796	562	47.0	48.0	0.785
0.70	1.40	0.70	960	1580	620	1270	47.0	48.0	1.023
0.01	1.40	1.39	164	1580	1416	872	47.0	48.0	0.890
TEST NO. 10, SIL-O-CEL NATURAL BRICK, DENSITY 27.3 LB. PER CU. FT., POROSITY 82.0 PER CENT (SPECIMEN A).									
0.01	0.70	0.69	88	474	386	281	19.0	19.3	0.652
0.70	1.40	0.70	474	815	341	649	19.0	19.3	0.748
0.01	1.40	1.39	88	815	727	452	19.0	19.3	0.697
0.01	0.70	0.69	112	717	605	415	30.0	30.6	0.658
0.70	1.40	0.70	717	1212	495	964	30.0	30.6	0.817
0.01	1.40	1.39	112	1212	1100	662	30.0	30.6	0.730
0.01	0.70	0.69	135	870	735	503	37.0	37.8	0.670
0.70	1.40	0.70	870	1461	591	1165	37.0	37.8	0.846
0.01	1.40	1.39	135	1461	1326	798	37.0	37.8	0.748
TEST NO. 11, SIL-O-CEL NATURAL BRICK, DENSITY 28.1 LB. PER CU. FT., POROSITY 85.6 PER CENT (SPECIMEN B).									
0.01	0.70	0.69	87	470	383	278	17.0	17.3	0.588
0.70	1.40	0.70	470	815	345	642	17.0	17.3	0.663
0.01	1.40	1.39	87	815	728	451	17.0	17.3	0.624
0.01	0.70	0.69	92	695	603	393	29.0	29.6	0.639
0.70	1.40	0.70	695	1210	515	952	29.0	29.6	0.760
0.01	1.40	1.39	92	1210	1118	651	29.0	29.6	0.695
0.01	0.70	0.69	102	948	846	525	43.0	43.6	0.672
0.70	1.40	0.70	948	1648	700	1298	43.0	43.6	0.823
0.01	1.40	1.39	102	1648	1546	875	43.0	43.6	0.740

CALCULATIONS

The method of calculating the thermal conductivity of Sil-O-Cel C-22 brick, for example, is as follows:

	TEST No. 1	TEST No. 2
Average temperature of heater plate, deg. Fahr.........	1245	1860
Indicated power (cut in circuit), watts................	66	110
Wattmeter circuit correction, per cent.................	+3.3	+4.1
Correction for loss in power supplied to heater plate, per cent (Fig. 3)................................	0	−3.0
True power supplied to center area, watts.............	68.2	111.1
Area of plate (inside dotted line Fig. 1)................	13.01 sq. in. or	
(Total area with two test specimens is twice the area of plate.)	0.09035 sq. ft	

$$\text{Thermal Conductivity,}^1\ k = \frac{Q}{2A} \times \frac{x_2 - x_1.}{T_2 - T_1}$$

where Q = quantity of heat in B. t. u. per hour = 3.412 × watts;
A = area of plate in square feet;
$x_2 - x_1$ = distance in inches;
$T_2 - T_1$ = temperature difference in degrees Fahrenheit.

$$\text{Therefore, } k = \frac{3.412 \times \text{watts}}{0.1807} \times \frac{x_2 - x_1}{T_2 - T_1} = 18.88 \times \text{watts} \times \frac{x_2 - x_1}{T_2 - T}$$

in B. t. u. per sq. ft. per hour per inch per 1° F.

The ratio $\frac{x_2 - x_1}{T_2 - T_1}$ was obtained from the average gradient curve and three calculations of k made to determine (1) the mean conductivity for each half of the gradient curve, and (2) the average conductivity of the whole specimen. These points determined the conductivity temperature curve which is approximately equivalent to the first derivative of the gradient curve. By making two or three equilibrium measurements more than one point at approximately the same temperature was obtained.

The results of these calculations on various types of insulating materials are given in Tables I and II. In the test on fire-clay brick with two specimens in series the mean conductivity was calculated for each section. (The temperature drop at the joints of these specimens was found to be 17 to 27° F. but the drop in calcined Sil-O-Cel was negligible as shown in Fig. 10.)

The results of the conductivity calculations for both fire-clay and red brick are given in Table III. The data are plotted in Figs. 18

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[1] The symbols used are recommended by E. F. Mueller, Report, Am. Soc. Refrigerating Engrs., Insulation Committee, p. 5 (Revised 1924).

to 21 inclusive to show the increase in conductivity with temperature, and the relationship of conductivity to density of the same material. The data for all the tests are summarized in Table IV for comparison.

TABLE II.—THERMAL CONDUCTIVITY OF INSULATING POWDER AND AGGREGATE.

Dimension x_1, in.	Dimension x_2, in.	Distance, $x_2 - x_1$, in.	Temperature T_1, deg. Fahr.	Temperature T_2, deg. Fahr.	Temperature Difference $T_2 - T_1$, deg. Fahr.	Mean Temperature, $\frac{T_1 + T_2}{2}$, deg. Fahr.	Watts Indicated	Watts Corrected	k, B.t.u. per sq. ft. per hour per inch per 1° F.
TEST NO. 12, SIL-O-CEL POWDER, DENSITY 16.0 LB. PER CU. FT. (SPECIMEN A).									
0.01	0.70	0.69	65	780	715	423	26.0	26.4	0.481
0.70	1.38	0.68	780	1270	490	1025	26.0	26.4	0.692
0.01	1.38	1.37	65	1270	1205	668	26.0	26.4	0.567
0.01	0.70	0.69	90	978	888	534	35.0	35.6	0.522
0.70	1.38	0.68	978	1554	576	1266	35.0	35.6	0.794
0.01	1.38	1.37	90	1554	1464	822	35.0	35.6	0.628
TEST NO. 13, SIL-O-CEL POWDER, DENSITY 18.5 LB. PER CU. FT. (SPECIMEN B)									
0.01	0.70	0.69	80	785	705	433	27.0	27.5	0.508
0.70	1.38	0.68	785	1272	487	1029	27.0	27.5	0.725
0.01	1.38	1.37	80	1272	1192	676	27.0	27.5	0.597
0.01	0.70	0.69	94	1052	958	573	39.0	39.2	0.533
0.70	1.38	0.68	1052	1685	633	1369	39.0	39.2	0.795
0.01	1.38	1.37	94	1685	1591	890	39.0	39.2	0.637
TEST NO. 14, SIL-O-CEL COARSE GRADE, DENSITY 18.0 LB. PER CU. FT. (SPECIMEN A).									
0.01	0.70	0.69	81	710	629	400	25.0	25.4	0.526
0.70	1.38	0.68	710	1150	440	930	25.0	25.4	0.741
0.01	1.38	1.37	81	1150	1069	616	25.0	25.4	0.615
0.01	0.70	0.69	84	974	890	529	39.0	39.7	0.581
0.70	1.38	0.68	974	1555	581	1265	39.0	39.7	0.878
0.01	1.38	1.37	84	1555	1471	820	39.0	39.7	0.698
TEST NO. 15, SIL-O-CEL COARSE GRADE, DENSITY 22.0 LB. PER CU. FT. (SPECIMEN B).									
0.01	0.70	0.69	83	678	595	381	26.0	26.5	0.580
0.70	1.38	0.68	678	1137	459	908	26.0	26.5	0.741
0.01	1.38	1.37	83	1140	1057	612	26.0	26.5	0.648
0.01	0.70	0.69	100	960	860	530	40.0	40.7	0.617
0.70	1.38	0.68	960	1568	608	1264	40.0	40.7	0.860
0.01	1.38	1.37	100	1568	1468	834	40.0	40.7	0.718

PRECISION OF MEASUREMENT

The following errors have been considered to establish the reliability of the above results:

Errors of Measurement:

Wattmeter.—The guaranteed accuracy of the Weston wattmeters is 0.25 per cent of full scale reading. Assuming the average power of one-third the full scale reading in the conductivity measurements the possible error in the meter reading would be 0.75 per cent.

Wattmeter Circuit Correction.—The correction for the error in the wattmeter readings due to the additional resistance when the meter is connected in the circuit was measured as previously described.

Power Loss in Heater Plate.—This loss in power due to the increase in electrical conductivity of Alundum at high temperatures was also measured and the correction applied.

TABLE III.—THERMAL CONDUCTIVITY OF FIRE-CLAY AND RED BRICK.

(Calcined Sil-O-Cel plates, $\frac{3}{8}$ in. thick between specimens and water-cooled plates.)

Dimnesion x_1, in.	Dimension x_2, in.	Distance $x_2 - x_1$, in.	Temperature T_1, deg. Fahr.	Temperature T_2, deg. Fahr.	Temperature Difference $T_2 - T_1$, deg. Fahr.	Mean Temperature, $\frac{T_1 + T_2}{2}$, deg. Fahr.	Watts Indicated	Watts Corrected	k, B.t.u. per sq. ft. per hour per inch per 1° F.
TEST NO. 16, FIRE-CLAY BRICK, FIRST QUALITY MISSOURI, DENSITY 120.5 LB. PER CU. FT., POROSITY 26.5 PER CENT									
0.03	0.78	0.75	571	662	91	617	44.0	44.0	6.85
0.88	1.78	0.90	691	797	106	744	44.0	44.0	7.06
0.03	0.78	0.75	886	1025	139	956	70.0	70.0	7.14
0.88	1.78	0.90	1062	1216	154	1139	70.0	70.0	7.73
0.03	0.78	0.75	1226	1426	200	1326	110.0	108.5	7.69
0.88	1.78	0.90	1471	1699	228	1585	110.0	108.5	8.09
0.03	0.78	0.75	1346	1562	216	1453	130.0	126.1	8.27
0.88	1.78	0.90	1611	1862	251	1737	130.0	126.1	8.54
TEST NO. 17, RED BRICK, HARD BURNED, DENSITY 131 LB. PER CU. FT., POROSITY 23.7 PER CENT.									
0.01[a]	2.22	2.21	332	993	661	662	120.0	120.0	7.58
0.01[a]	2.22	2.21	458	1292	834	875	160.0	160.0	8.01
0.01	2.22	2.21	998	1335	337	1167	78.0	78.0	9.67
0.01	2.22	2.21	1406	1947	541	1676	154.0	148.0	11.42
TEST NO. 18, RED BRICK, SOFT BURNED, DENSITY 109.5 LB. PER CU. FT., POROSITY 38.3 PER CENT.									
0.02[a]	2.17	2.15	247	714	467	480	50.0	50.0	4.34
0.02	2.17	2.15	565	930	365	747	40.0	40.0	4.45
0.02	2.17	2.15	741	1184	443	962	55.0	55.0	5.04
0.02	2.17	2.15	846	1377	531	1111	69.0	69.0	5.28
0.02	2.17	2.15	1118	1810	692	1464	98.0	95.6	5.61

[a] Insulating plates not used.

Voltage Variation.—The voltage of the 60-cycle power line was subject to variation at times during the conductivity measurements. The correction for these fluctuations could only be made by adjusting the power every five to ten minutes during the time of thermal equilibrium. Under such conditions the mean power input was not necessarily the same as the adjusted power. If the voltage were gradually increasing the mean power would be high and *vice-versa*, but usually the voltage fluctuated in both directions. The maximum variation in power due to voltage changes was about 2 per cent so the error in the mean power input would be about 1 per cent.

Location of Thermocouples.—The position of the thermocouple junctions embedded in the specimens was measured with calipers and a depth gage to less than 0.01 in. The minimum thickness used in the calculations was about 0.7 in. so the greatest error due to these measurements is about 1 per cent.

TABLE IV.—SUMMARY OF THERMAL CONDUCTIVITY DATA.

Material	Test	Density, lb. per cu. ft.	Porosity, per cent	Thermal Conductivity, B.t.u. per sq. ft. per hour per inch per 1° F.	
				500° F.	1500° F
Sil-O-Cel C-22 Brick	No. 2	40.0	75.0	1.73	2.18
Sil-O-Cel C-22 Brick	No. 3	36.0	75.7	1.62	2.04
Average	....	38.0	75.4	1.68	2.11
Special Sil-O-Cel Brick	No. 4	33.8	76.7	1.13	1.45
Sil-O-Cel Super Brick	No. 5	43.9	72.6	1.83	2.27
Sil-O-Cel Super Brick	No. 6	40.7	76.7	1.58	2.10
Average	....	42.3	74.6	1.71	2.19
Sil-O-Cel C-3 Concrete (Dehydrated at 800° F.)	No. 7	57.6	65.2	2.02	2.75
Calcined Sil-O-Cel Brick	No. 8	28.1	83.9	0.725	0.895
Calcined Sil-O-Cel Brick	No. 9	31.8	84.0	0.785	1.095
Average	....	30.0	84.0	0.755	0.995
Sil-O-Cel Natural Brick	No. 10	27.3	82.0	0.700	0.925
Sil-O-Cel Natural Brick	No. 11	28.1	85.6	0.650	0.875
Average	....	27.7	83.8	0.675	0.900
Sil-O-Cel Powder	No. 12	16.0	....	0.500	0.870
Sil-O-Cel Powder	No. 13	18.5	....	0.530	0.850
Average	....	17.3	....	0.515	0.860
Sil-O-Cel Coarse Grade	No. 14	18.0	....	0.565	0.970
Sil-O-Cel Coarse Grade	No. 15	22.0	....	0.610	0.930
Average	....	20.0	....	0.588	0.950
Fire-clay Brick, First Quality Missouri	No 16	120.5	26.5	6.65	8.15
Red Brick, Hard Burned	No 17	131.0	23.7	6.85	10.70
Red Brick, Soft Burned	No. 18	109.5	38.3	4.20	5.75

Measurement of Temperature.—In calibrating the thermocouples with the standard rare metal couple the maximum observed variation was about 0.33 per cent. Therefore, the error in the temperature differences in each half of the specimens would be from 0.67 to 1 per cent.

Area of Power Measurement.—The area of the heater plate inside the potential leads was accurately measured with a carbon copy of the plate and a planimeter. The error in this area shown in Fig. 1 is probably negligible.

Errors in the Method:

Direction of Heat Flow.—In all conductivity determinations the temperature gradient should be measured in a direction perpendicular to the plane of the area under test. With the plate type of apparatus the guard ring is provided to overcome the edge losses and, therefore, prevent lateral flow of heat. In order to obtain equilibrium the edge heater usually required 20 to 60 per cent higher current than the center heater. In the use of a similar test apparatus Heilman[1] found that it was not possible to obtain a uniform temperature over the face of the heating plate on account of excessive flux of heat from the edge to the center.

In a number of the thermal conductivity measurements a series of tests were made to determine the uniformity of the plate temperature and the lateral heat flow by placing auxiliary thermocouples on the hot and cold surfaces 2 in. from the center of the specimen. The mean temperature difference between the couples on the hot surfaces for 18 observations was found to be 1° F. which is within the precision of the temperature control on the differential couples during equilibrium. The mean temperature difference between the couples on the cold surfaces for 47 observations was found to be 24° F. The temperature 2 in. from the center was higher than that at the center in about one-half the measurements, which indicates that there was no lateral flow of heat in any one direction. Similar measurements were made in one test (Table III, Test No. 17) on a more conductive material of greater thickness, with and without the use of the $\frac{3}{8}$-in. Sil-O-Cel insulating plates. The mean temperature on the cold surfaces 2 in. from the center was 20° F. lower than that at the center when the plates were used, but this mean temperature was 95° F. higher when the insulation was removed. This indicates that the lateral heat loss from comparatively thick (2.2 in.) and highly conductive materials is not appreciable after the heat flow is reduced by the added resistance of the insulation.

The lateral flow of heat from a plate type of apparatus with a 2-in. guard ring was calculated by Van Dusen.[2] With a 1-in. specimen there was theoretically no lateral heat loss from the center area at a point 1.18 in. from the edge. The 2-in. guard ring on the test apparatus was evidently ample for preventing lateral flow from specimens $1\frac{1}{2}$ in. or more in thickness.

The conduction of heat along the thermocouples would not affect the heat flow appreciably since the wires were placed approximately parallel to the surfaces of the specimen in the isothermal lines. The change in structure of the specimens due to the thermocouple holes filled with powder would not alter the heat flow appreciably since the volume of these holes is negligible compared to the volume of specimen under test.

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[1] R. H Heilman, *Transactions*, Am. Inst. Chemical Engrs., Vol. 18, p. 283 (1926).

[2] M. S. Van Dusen, *Journal*, Am. Soc. Heating and Ventilating Engrs., Vol. 26, p. 625 (1920).

Equal Division of Heat.—With this type of apparatus and with identical specimens it is assumed that the heat from the plate divides equally between the specimens. It is shown on the gradient curves (Figs. 4 to 17) that the temperatures in both specimens checked quite

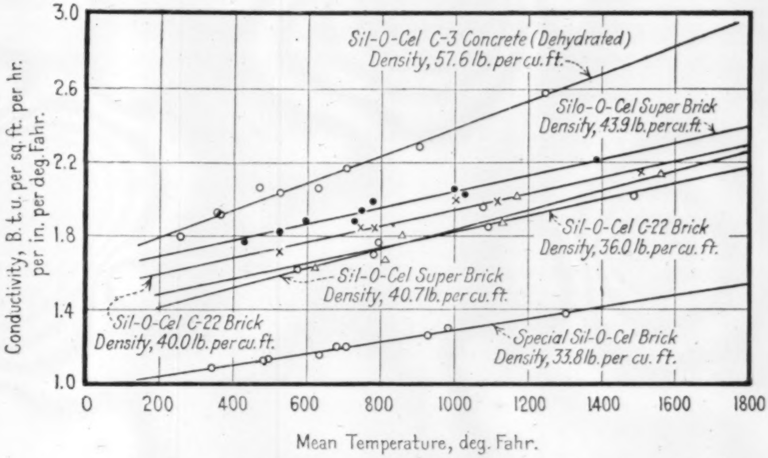

FIG. 18.—Thermal Conductivity of High Temperature Insulating Brick and Concrete.

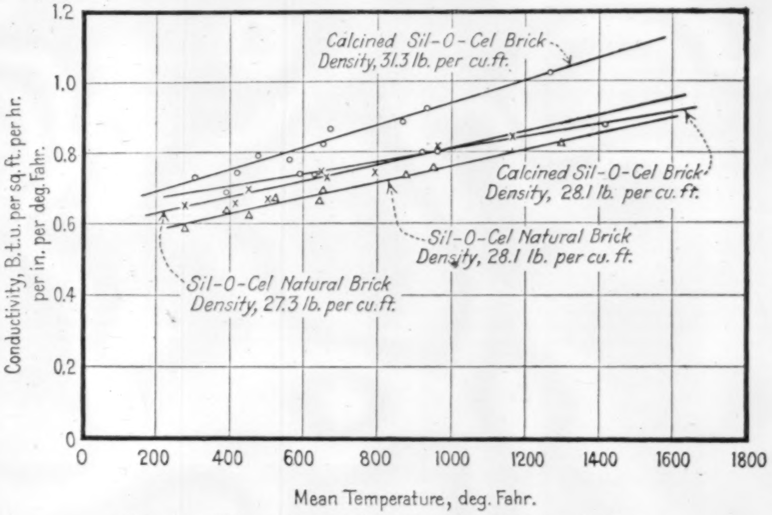

FIG. 19.—Thermal Conductivity of Insulating Brick.

closely, which indicates practically equal division of heat. In all the calculations these average gradients were used, so this error is negligible.

Calculations.—The errors in drawing the gradient curves through the observed points introduce some error in the conductivity calculations. With two or three interior temperature measurements this error is probably less than 1 per cent.

As shown in Figs. 18 to 21, inclusive, the calculated points deviate from the mean conductivity curve. The amount of this deviation gives an estimate of the precision of the measurements. The average percentage deviations of the points for three representative tests were calculated and divided by the square root of the number of equilibrium tests to determine the reliability of the results. This was found to be less than 1 per cent.

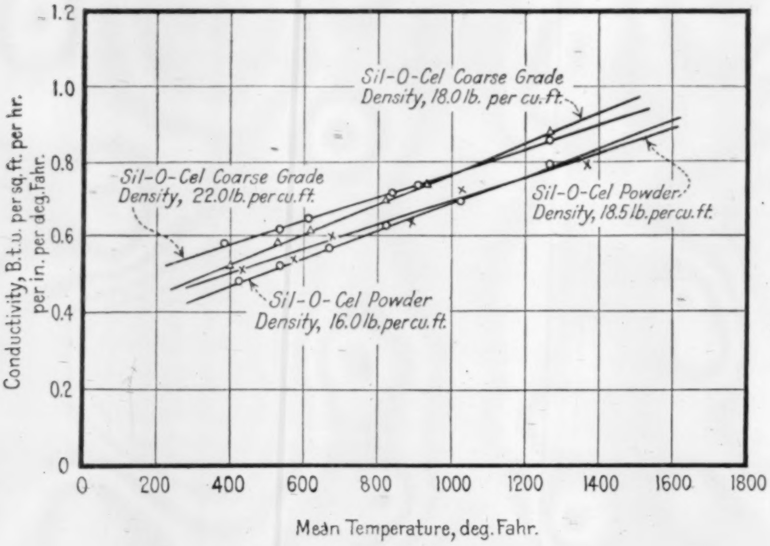

FIG. 20.—Thermal Conductivity of Insulating Powder and Aggregate.

Overall Precision of Measurements.—The appreciable errors in the conductivity measurements are as follows:

Wattmeter	0.75 per cent
Voltage Variation	1 per cent
Location of Thermocouples	1 per cent
Measurement of Temperatures	1 per cent
Mean Gradient Curve	1 per cent
Deviation Measure	1 per cent

From these estimates of the residual and accidental errors the probable measure of the reliability of the conductivity results is equal to the square root of the sum of the squares or about 2.5 per cent.

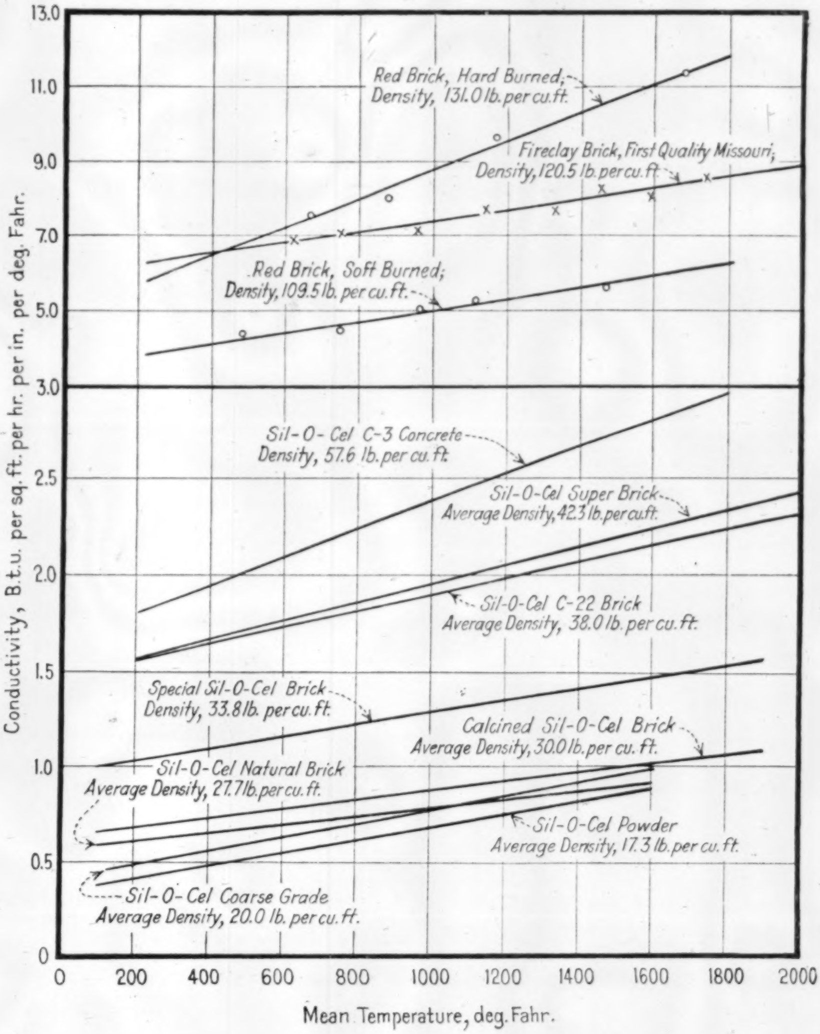

FIG. 21.—Comparative Thermal Conductivities of Various Materials.

SUGGESTED STANDARD TEST METHODS

It would be desirable to use one type of apparatus for making standard thermal conductivity measurements on all insulating and refractory materials. The American Society of Heating and Ventilating Engineers have accepted tentatively the guarded plate method described by Van Dusen[1] as the standard test for measuring the conductivity of building materials. The apparatus recommended for this is very satisfactory for low temperature tests but it is not adapted for high temperature measurements.

With the three test methods described the same apparatus can be used for measuring accurately the thermal conductivity of insulating and refractory materials at mean temperatures up to about 1800° F. This apparatus, of course, cannot be used for testing insulation with curved surfaces, for example, pipe coverings. In case of all insulation for curved surfaces, special flat specimens of the material could be prepared for the standard test.

It is, therefore, suggested that a guarded flat plate apparatus similar to that described, be used for standard thermal conductivity measurements. The heater plates to be used by various investigators might preferably be standardized (perhaps at the Bureau of Standards) as to effective plate area, the power losses in the plate, the accuracy of the differential couples, etc. With this standard equipment it is suggested that the thermal conductivity measurements be made on 1.5-in. specimens as described for (1) low-temperature insulators, (2) medium to high-temperature insulators, and (3) refractory materials.

SUGGESTED STANDARD METHODS OF EXPRESSING RESULTS

Various methods have been used to express the relationship of thermal conductivity to temperature. The most common methods and the accuracy of each are discussed by McMillan.[2] In these methods the conductivity is expressed as a function of:

1. Temperature difference between surfaces;
2. Temperature of hot surface;
3. Temperature difference between the hot surface and room temperature;
4. Mean temperature.

Since the thermal conductivity of any material is a function of temperature, this property is expressed accurately as the conductivity at certain temperatures. It has been shown by Hering[3] and Barrett[4] that the average conductivity over the entire thickness of a material is equal to the true conductivity at the arithmetical mean of the two surface temperatures, provided that the conductivity is a linear function of temperature. McMillan[1] has shown that even if the conductivity varies as the square of the temperature, the error due to taking the conductivity at the arithmetical mean of the surface temperatures is usually less than 1 per cent. It is, therefore, suggested that all conductivities determined by standard tests be expressed in terms of the *mean* temperatures between the surfaces or sections of the test specimens.

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[1] M. S. Van Dusen, *Journal*, Am. Soc. Heating and Ventilating Engrs., Vol. 26, p. 625 (1920).
[2] L. B. McMillan, *Transactions*, Am. Soc. Mechanical Engrs., Vol. 48, p. 1269 (1926).
[3] C. Hering, *Transactions*, Am. Electrochemical Soc., Vol. 21, p. 520 (1912).
[4] L. L. Barrett, *Transactions*, Am. Soc. Mechanical Engrs., Vol. 44, p. 315 (1922).

CONCLUSIONS

The modified guarded hot plate type of apparatus can be used for making thermal conductivity measurements at mean temperatures up to approximately 1800° F.

The heater plates must be calibrated to correct for excessive power losses due to the increased electrical conductivity of Alundum at high temperatures.

The same apparatus can be used to measure the thermal conductivity of (1) organic or hydrated insulators, (2) medium to high-temperature insulators, and (3) refractory materials.

The reliability of the conductivity measurements is estimated to be about 2.5 per cent.

The apparatus and the methods described appear to be of sufficient accuracy and reliability to be given consideration for specifications and the establishment of standards.

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[1] L. B. McMillan, *Transactions*, Am. Soc. Mechanical Engrs., Vol. 48, p. 1269 (1926).

THE TEST FOR VOLATILE COMBUSTIBLE MATTER AS APPLIED TO COAL-TAR PITCH

By S. R. Church[1] and W. H. Fulweiler[2]

Synopsis

The paper gives a short history of the genesis and development of the test for volatile combustible matter as applied to tar pitches, with particular reference to pitch used as fuel or in the manufacture of fuel. Variables in the test as practiced in the United States and in Europe are shown in tabular form. It is shown that there are four principal variables in the method of which two relate to the size and type of crucible used, one to the kind of heat and time of heating, and the other and most important to the use or non-use of preliminary heating. It is shown that the use of preliminary heating as recommended in the method for lignites and sub-bituminous coals gives variable results at a lower order than found when it is not used. It is concluded that the standard A.S.T.M. method for coal or possibly a slight modification of it is satisfactory for testing pitch, and that the use of preliminary heating should be definitely avoided.

Introduction

The volatile combustible test is well known as one of the methods used in making "proximate analysis" of coal. The test method has been an A.S.T.M. standard for some time.[3] The present standard is the outcome of numerous revisions of a method first adopted as an American Standard in 1899 by a committee of the American Chemical Society.[4] The report of this committee refers to a method described by Muck[5] which is very similar to the present coal method. While this is primarily a test for coal and coke, it has been used quite extensively on artificial and natural bitumens. It is the purpose of this paper to deal only with its use in testing tar pitches.

Tar pitches are used for a variety of purposes that need not be mentioned here. The Society, through its Committees D-4 on Road and Paving Materials and D-8 on Bituminous Waterproofing and Roofing Materials, has standardized numerous methods for testing pitches. Some of these methods, in particular the tests for melting point[1] and for insoluble matter (free carbon),[2] are applicable to pitches used as fuel or in the manufacture of fuel, but neither this Society nor any other American national body has adopted a method for determining volatile combustible matter in pitch.

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[1] Consulting Chemist, New York City.

[2] Chemical Engineer, United Gas Improvement Co., Philadelphia, Pa.

[3] See Standard Methods of Laboratory Sampling and Analysis of Coal and Coke (D 271 - 27), 1927 Book of A.S.T.M. Standards, Part II, p. 535.

[4] *Journal*, Am. Chemical Soc., Vol. 21, p. 1122.

[5] "Chemie der Steinkohlen," 2d Ed., 1891, p. 10.

Dehydrated coal tar contains about 80 to 90 per cent of volatile combustible matter. The residual products from distillation of coal tar, ranging from soft pitch down to the hardest pitches commercially made, contain from 80 per cent to about 40 per cent of volatile combustible matter.

Tar in every form from crude tar to hard pitch has been used as a fuel. Both tar and pitch are used as liquid fuel and the harder pitches as pulverized fuel. Pitch used in the manufacture of fuel briquets may be used either in liquid form (hot) or as a ground solid. In either case it is mixed with ground coal to act as a binder, and compressed into hard formed cakes of varying sizes and shapes. The pitch constitutes from 6 to 9 per cent by weight of the material used.

The manufacture of fuel briquets has not yet attained to a very large tonnage in the United States, but in Europe it has long been an industry of considerable magnitude, located chiefly in Wales and in Central and Western Europe. Pitch is used almost exclusively in the European briquet industry but in the United States other binders including oil asphalt, sulfite liquor and various other substances are also used.

It seems natural that since briquet pitch enters into a fuel largely composed of coal, the common analytical methods for volatile combustible matter should be applied to it; and in fact this is so, and purchase specifications for pitch, especially for export shipment to Europe, generally include upper and lower limits of "volatile combustible matter" within the general range of 50 to 70 per cent. It is not uncommon to find upper and lower limits with a range of 10 per cent.

METHODS PROPOSED

The literature on testing methods for tar products contains several references to this test. These indicate considerable variation in details. Constam and Rougeot in "Über die Bestimmung der Koks Ausdente von Bindermittel bei Steinkohlen Briketts"[1] refer to Brookman's modification of Muck's method, in which a platinum crucible having a lid with a central hole 2 mm. in diameter is used, the crucible held 6 cm. above the burner and heated "as long as the flames issue." This method is also referred to by Lunge.[2] Malatesta in his book on Coal Tars and Their Derivatives[3] describes the testing of pitch for fixed carbon: "One gram in platinum crucible 20 to 30 cc. capacity, covered with lid. Heat for 7 minutes over flame of Bunsen burner taking care to place the crucible 6 to 8 cm. above the burner and to use a flame 20 cm. high."

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[1] See Standard Method of Test for Softening Point of Tar Products (Cube-in-Water Method) (D 61 – 24), 1927 Book of A.S.T.M. Standards, Part II, p. 501.

[2] See Standard Method of Test for the Determination of Bitumen (D 4 – 27), 1927 Book of A.S.T.M. Standards, Part II, p. 484.

Warnes in his book on Coal-Tar Distillation[4] says there are several methods but no standard.

TABLE I.—COMPARISON OF METHODS FOR VOLATILE COMBUSTIBLE MATTER IN COAL-TAR PITCH.

	Church	Allen	Weiss	A.S.T.M.	Committee on Tar Testing Methods	Spilker
Country	U. S. A.	U. S. A.	U. S. A.	U. S. A.	England	Germany
Year	1911	1912	1918	1927	1928	1928
Crucible	Platinum	Platinum	Platinum	Platinum	Platinum	Platinum
Height, mm	......	......	......	30 - 35	35 - 40	35
Diameter, mm	......	......	......	25 - 35	24 - 25 at bottom	22 - 23
Capacity, cc	20	......	20	10 - 20	......	......
Sample, g	1	1	1	1	1	1
Method of heating	Flame	Flame	Flame	Furnace or flame	Meker No. 4	Flame
Height, cm	20	......	20	......	......	18
Distance, cm	6 - 8	......	6 - 8	......	10	......
Temperature	......	......	......	950° C.±20° C.	......	......
Time, minutes	7	15	7	7	3	......
Lid	Close fitting	......	Tight	Close fitting	2-mm. hole	2-mm. hole
Special Directions	[a]	......	[b]	[c]	......	[d]

[a] Preliminary heat for a few minutes until danger of foaming is passed.
[b] The tar shall be heated gently until the tendency to foam is passed.
[c] Peat, sub-bituminous coal and lignite must be subjected to preliminary gradual heating for five minutes.
[d] Heat until no ignitable gases are given off.

W. G. Adam[5] of London in a general plea for standardization of tar products testing methods says, "The volatile matter content is determined by heating the powdered pitch in a platinum crucible in a similar manner to the standard test for coal. The size of the crucible, height of flame and particularly the rapidity with which the initial heating of the pitch takes place will, if varied, give considerably different results. It is suggested that the standard method adopted should closely follow the well-known Brookman-Muck test." (It was this paper by Adam that precipitated the formation of a comprehensive Committee on Tar Testing Methods, which is now at work, and which has been in communication with several of the Society's committees on various methods.)

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[1] *Zeitschrift Angewandte Chemie*, 1904, p. 845.

[2] "Coal Tar and Ammonia," 5th Ed., 1916.

[3] "Coal Tar and Their Derivatives (Italian)," London, 1920, p. 508.

[4] "Coal-Tar Distillation," 3d Ed., London, 1923, p. 388.

[5] "A Plea for Standardization of Analytical Methods in Coal-Tar Products Specifications," Report of Joint Committee on Tar Conference, *Journal*, Soc. Chemical Industry, November, 1926, p. 14.

In American literature the earliest reference appears to be Church.[1] In publishing The Barrett Company's methods of testing tar products, he described the coal method as applied to crude coal tar, with a note calling for preliminary heating "for a few minutes" to avoid foaming. Abraham[2] gives a method for pitch which is the same as the coal method with no precaution as to heating.

Allen[3] describes a method involving 15 minutes' preheating and ending with 10 minutes over a blast lamp. This method is obviously far out of line with any of the others described.

In Table I we have recapitulated the detailed requirements of the American, English and German methods, together with the method prescribed for coal in the Society's Standard Methods of Laboratory Sampling and Analysis of Coal and Coke (D 271 - 27).[4]

TABLE II.—COMPARISON OF VOLATILE TESTS ON PITCH BY UNITED STATES AND ENGLISH METHODS.

	Volatile Combustible Matter, per cent			
	Sample No. 1	Sample No. 2	Sample No. 3	Sample No. 4
A.S.T.M. Method (electric furnace)	71.5	69.8	62.0	69.7
English Method (4-mm. hole in lid of crucible)	69.3	68.0	61.3	68.8

The English method as shown in the table will shortly be adopted by the Committee on Tar Testing Methods, according to a very recent private communication from London. The details of the German method have been taken from a letter from Mr. Spilker. It will be seen that these methods are now practically alike; both use a lid with 2-mm. hole, and neither requires preheating.

Preliminary Heating:

In a recent litigation involving a cargo of pitch, tests were made which showed clearly the need for better agreement regarding the details of carrying out this test, and especially the matter of preliminary heating. It was brought out by comparison of results obtained by different analysts, that by using preliminary heating on the one hand, and none on the other, results differing by 10 per cent were obtained on the same sample of pitch.

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[1] *Journal of Industrial and Engineering Chemistry*, Vol. 3, p. 297 (1911).

[2] "Asphalts and Allied Substances," by Herbert Abraham.

[3] *Commercial Organic Analysis*, Vol. 12, p. 55.

[4] 1927 Book of A.S.T.M. Standards, Part II, p. 535.

The discussion that followed developed considerable difference of opinion as to the merit of preliminary heating and the reason for the low results obtained when it is practiced.

TABLE III.—VOLATILE COMBUSTIBLE TESTS SHOWING VARIATION DUE TO TIME AND MANNER OF PRELIMINARY HEATING.

	VOLATILE COMBUSTIBLE MATTER, PER CENT
Preheating with small stationary flame:	
for one minute	67.4
for two minutes	66.2
for three minutes	65.9
for four minutes	64.6
Preheating with full flame applied to bottom and sides of crucible:	
for 20 seconds	72.1
for 40 seconds	69.4
for 60 seconds	68.2

Before giving our views on this question, we will show the results of some tests made for the purpose of illustration:

Table II gives comparative tests on four samples of pitch by the A.S.T.M. method for coal using an electric furnace, and an English method with a 4-mm. hole in the crucible lid. No preheating was used in either series of tests. (The English method has since been modified with respect to the size of hole in the lid, probably to conform to general European practice.)

TABLE IV.—COMPARISON BETWEEN A.S.T.M. AND ENGLISH METHOD, PRELIMINARY HEATING FOR TWO MINUTES OVER A SMALL FLAME, AND USE OF SAND.

	Volatile Combustible Matter, per cent			
	Sample No. 1	Sample No. 2	Sample No. 3	Sample No. 4
A.S.T.M. method	71.5	69.8	62.0	69.7
English method	69.3	68.0	61.3	68.8
With preheating	54.9, 63.0[a]	60.6, 62.9[a]	55.0	58.7
Employing 10 g. ignited silica sand over sample	64.0	61.8	56.7	62.0

[a] Two values through variations in time of application and size of flame.

The average difference of 1.4 per cent between the two series may be partly accounted for by the shorter time of heating in the English method.

Table III gives tests on one sample, with preliminary heating, showing variation due to changing the time of preheating and the manner of applying the preheat. There is a difference of 7.5 per cent between the high and low results in this series.

Table IV shows again the results on four samples, comparing the A.S.T.M. method for coal, the English method, preheating with variation of time and size of flame (on Samples Nos. 1 and 2) and finally a method that has been suggested for highly decrepitating lignites, etc., using a silica sand cover over the sample.

The extreme difference in results on one sample is 16.6 per cent.

The question of preliminary heating then comes down to:

1. Is there a good reason for using preheating in testing pitch?

2. If preheating is used as a precautionary means against foaming due to moisture in sample, how and for how long should it be applied?

The authors are definitely of the opinion that:

1. There is no need for preheating, at least in reasonably dry samples; but that

2. Preheating as a precautionary matter does not seriously affect the results if applied for not more than 20 to 30 seconds by playing the flame around the sides and top of the crucible.

Pitch behaves in an entirely different manner during this test than sub-bituminous coals, lignites, etc. The latter carry high moisture content and if suddenly exposed to full heat pop violently. Pitch melts completely during the first few seconds of heating. It could be carried out of the crucible mechanically only by foaming over, and observation shows that this does not take place when no preheating is used.

Pitch on the other hand is sensitive to thermal decomposition, particularly if heat is slowly applied at the bottom of the vessel, with upper walls and lid acting as a condenser for the vapor evolved. Therefore, by varying the time of preheating and by avoiding the application of heat on the upper sides of the crucible, the vapors can be decomposed and the fixed carbon content increased to the extent of 5 to 15 per cent if not more.

Mr. Adam in a letter to one of the authors says: "No doubt in the course of your work you will have recognized that the matter is a somewhat different one from the estimation of volatile matter in coal, where one of the main difficulties is to avoid combustion of fixed carbon. In the case of pitch, the difficulty is to avoid cracking the vapors and depositing carbon within the crucible, and hence a short high heat quickly applied is adopted." This entirely confirms our own opinion.

CONCLUSIONS

Summing up, the principal variables in the methods used for testing pitch for volatile combustible matter are:

1. Size of crucible;
2. Type of lid;
3. Kind of heat (burner or furnace) and time of heating;
4. Use or non-use of preliminary heating.

The first three variables, within the range of the specifications quoted, do not affect the results to an important degree.

We consider the standard A.S.T.M. coal method as satisfactory for testing briquet pitch. If any modification of the method is to be made, for convenient and expeditious work, we favor using a lid with a small circular orifice and we favor a shorter heating time, preferably 3 minutes, to conform with English practice.

Either the burner or the electric furnace is a convenient and satisfactory method of heating. The latter is more easily controlled as to temperature. The use of a manometer to control gas pressure seems desirable when burners are used.

Preheating is unnecessary, and if applied in the manner described, and as recommended for use with lignites and certain coals, it seriously alters the results of the test.

THE DEVELOPMENT OF AN APPARATUS FOR WEAR TESTS ON FLOORING MATERIALS[1]

D. W. KESSLER[2]

SYNOPSIS

The wearing qualities of flooring materials have heretofore received very little attention by testing engineers and none of the testing devices in present use seem well adapted to such determinations. Due to the outstanding need for data on this property of the more permanent types of flooring, the author has worked out a simple abrasion apparatus in which the various testing conditions can be closely controlled, and a series of studies have been made to determine its adaptability to the purpose in view. Measurement of the actual wear on several materials under service conditions have been made and compared with the results of test on the same materials with the apparatus. Results of abrasion tests are given on 47 marbles, 15 limestones, 4 sandstones, 14 slates and 11 miscellaneous materials.

INTRODUCTION

A need for information concerning the wearing qualities of various flooring materials is forcefully shown in many high class buildings, particularly where the architect has created beautiful decorative designs by combining materials of different color tones. It frequently happens that the different materials possess such different degrees of resistance to wear that the surface becomes very uneven and repair is necessary. The object of the research described herein was to develop a suitable means of comparing the wearing qualities of various flooring materials and to establish data on the more permanent types.

CONSIDERATION OF METHODS

Various methods used for determining the hardness of rocks, minerals and metals have been considered, but the only apparatus manufactured in this country which seems to approach the conditions of wear on floors is the Dorry apparatus,[3] sometimes used in testing road materials. The Dorry apparatus employs a cylindrical specimen 25 mm. in diameter which is abraded against a steel disk under a weight of 1250 g. with crushed quartz graded between No. 30 and No. 40 screens. One thousand revolutions of the disk are made and the hardness value is expressed as $H = 20 - \frac{1}{3}W$ where W is the abrasion loss in grams. It was not considered to be well adapted for testing floor materials on account of the shape of specimen and the means of holding the specimen did not appear to be entirely satisfactory. Therefore it was decided to construct a simple abrasion apparatus in which the various test conditions could be closely controlled and determine if the results of such tests afford a reliable criterion for judging wearing qualities of flooring materials.

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[1] Published with the approval of the Director of the Bureau of Standards of the U. S. Department of Commerce, Washington, D. C.

[2] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

[3] Described in U. S. Department of Agriculture *Bulletin 44*, p. 7.

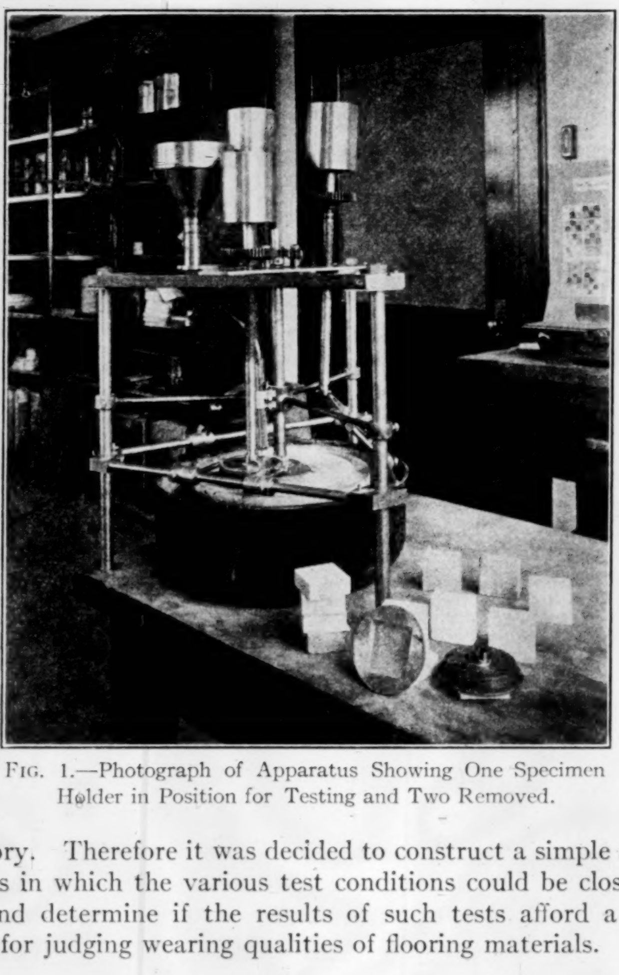
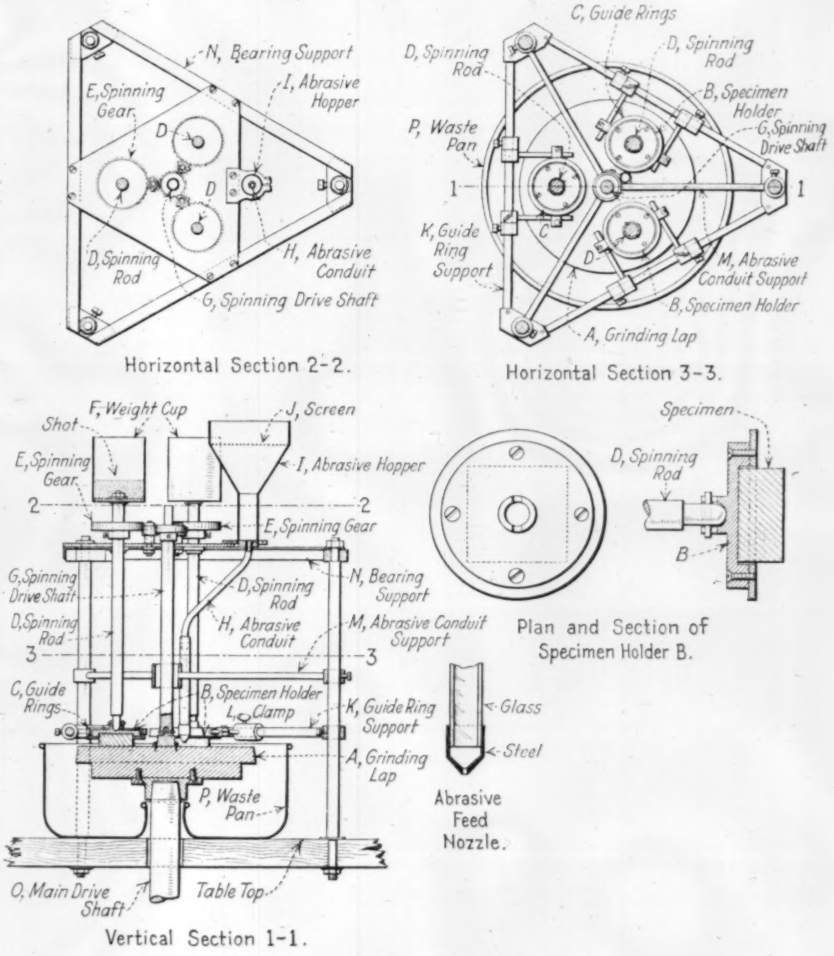


FIG. 1.—Photograph of Apparatus Showing One Specimen Holder in Position for Testing and Two Removed.

DESCRIPTION OF APPARATUS

The apparatus illustrated in Figs. 1 and 2 is the result of several modifications of one started by Mr. R. N. Young at the Bureau of

Standards about six years ago. It consists of a grinding lap, a means of rotating the specimens on the lap, and a means of feeding the abrasive. The greatest difficulty in developing the apparatus was to secure a satisfactory distribution of the abrasive throughout the test and prevent the accumulation of abraded material on the lap. The scheme of rotating square slab specimens in a direction opposite to that of the lap and feeding the abrasive through a small orifice near the center of the lap gives fairly satisfactory conditions in this


N, Bearing Support
I, Abrasive Hopper
E, Spinning Gear
D
D
D, Spinning Rod
H, Abrasive Conduit
G, Spinning Drive Shaft
C, Guide Rings
D, Spinning Rod
D, Spinning Rod
B, Specimen Holder
G, Spinning Drive Shaft
P, Waste Pan
1
1
K, Guide Ring Support
C
D
M, Abrasive Conduit Support
B, Specimen Holder
A, Grinding Lap
F, Weight Cup
Shot
J, Screen
E, Spinning Gear
I, Abrasive Hopper
2
2
E, Spinning Gear
G, Spinning Drive Shaft
D, Spinning Rod
D, Spinning Rod
N, Bearing Support
H, Abrasive Conduit
M, Abrasive Conduit Support
3
3
C, Guide Rings
B, Specimen Holder
L, Clamp
K, Guide Ring Support
A, Grinding Lap
P, Waste Pan
O, Main Drive Shaft
Table Top
Specimen
D, Spinning Rod
B
Glass
Steel

Horizontal Section 2-2.

Horizontal Section 3-3.

Vertical Section 1-1.

Plan and Section of Specimen Holder B.

Abrasive Feed Nozzle.

FIG. 2.—Plan and Sections of Apparatus.

respect. It also overcomes friction in the holder guides and hence variable pressure on the specimens as well as avoiding the possibility of wedging or binding of the holder.

The $\frac{3}{32}$-in. orifice placed $\frac{3}{8}$ in. above the lap allows the abrasive to build up a circular heap as the lap rotates. This is within reach of the corners of the specimens which rake out and distribute the grit as the specimens rotate. This process continually moves fresh grit from the center of the lap outward as the used grit and abraded material is forced over the edge of the disk. Three specimen holders, *B*, are provided which rotate at one-half the speed of the lap. The weight cups, *F*, at the top of the rods, *D*, are provided to enable one to place any desired load on the specimen. The total load used in the tests was two kilograms for each specimen. A correction was made in the computations for the weight of the specimen, since this means was found more expeditious than to adjust the superimposed weights. The specimen holder guide rings, *C*, are placed near the grinding surface in order to overcome, in so far as possible, the tendency of the specimen to tip and bear with greater force on one side than the other. The guide ring is clamped in place on the supporting rods, *K*, by means of the thumb screw, *L*, so its weight is not allowed to bear on the specimen. The specimen, which is a slab 2 in. square by $\frac{1}{2}$ in. or more in thickness, fits loosely in a square recess in the holder. The rods, *D*, are rounded on the ends and set loosely in a socket in the holder. Rotation is transmitted to the holder by means of a pin near the end of the rod which sets into a groove in the top of the holder. This connection between the rotating rod and specimen holder acts like a universal joint and causes the specimen to bear uniformly on the lap whether the specimen is of uniform thickness or not. It also permits the specimen to be easily inserted in place or removed.

PRELIMINARY EXPERIMENTS

Numerous experiments were made before a test procedure was decided upon. One problem was to determine the most satisfactory abrasive. Three well-known grits were tried, namely, silicon carbide, artificial corundum, and emery. Different grades of fineness, namely, Nos. 60, 70, 80, 90 and 100 were tried. It was found that No. 60 abraded faster and produced a surface more nearly like that of a worn floor than the finer grades. Experiments were made with these three abrasives by testing them against the same specimens of stone to determine their rates of abrasion and loss of abrading action due to repeated use. Silicon carbide and artificial corundum of No. 60

grade were used in these experiments, but since this size of emery could not be obtained the next finer grade was used. In these experiments six specimens of marble varying in hardness from soft to hard were tested with each grit. The amount of wear on the specimens afforded a comparison of the abrading action of the different grits. After each test the grit was placed on a No. 100 sieve and washed

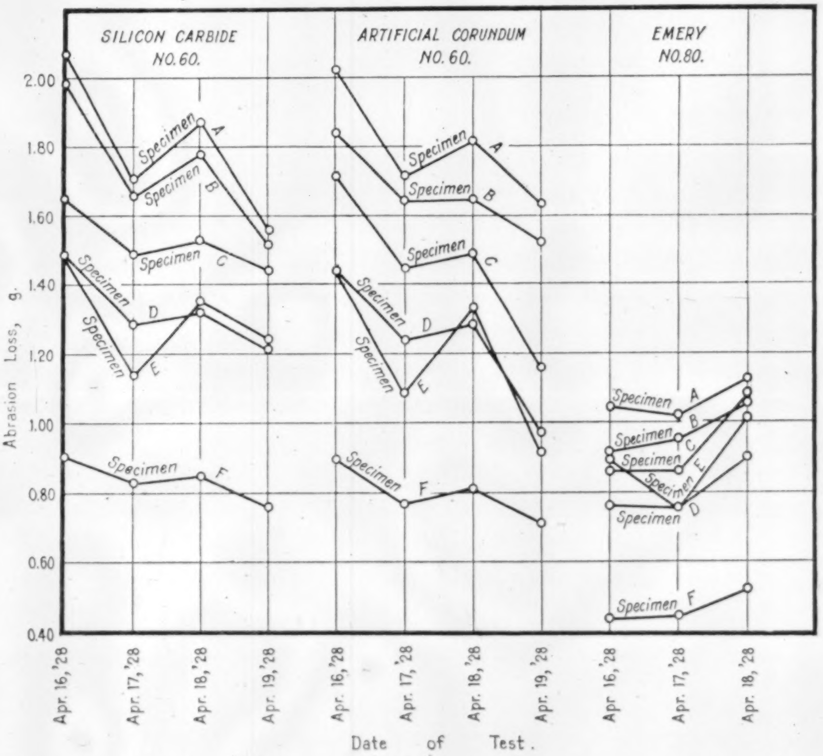


FIG. 3.—Repeat-Tests on Six Specimens of Stone with Different Abrasives, Showing Decrease in Abrading Action Due to Repeating with the Same Sample of Abrasive.

free of marble dust. It was also found that a considerable amount of the grits themselves was washed through the sieve. This was believed to be due to crumbling of the grains and to afford an indication of brittleness. Measurements of the amounts passing the sieve were also made for comparative purposes. After the used grits were cleaned and dried they were used over again on the same specimens in order to determine the decrease in abrading action. Four tests were thus made with silicon carbide and artificial corundum but the emery crumbled so fast that only three tests could be made with the sample. The results of these experiments are shown in Fig. 3. It will be noted that one test on each grit was made on one day and the process repeated on each of three succeeding days. On the third day instead of the abrading action being reduced it increased quite appreciably over that noted in the second set of tests. This increase was quite consistent for all grits and all materials. This effect on specimen *E*, a serpentine,[1] is shown to be more pronounced than on any of the others. Since the third set of tests was made on a rainy day it was assumed that the high humidity of the laboratory air was the cause of the increased abrading action. These experiments indicated that the first two abrasives were practically of equal value for the purpose while emery was inferior. The amounts of the first two crumbling and passing the No. 100 sieve were practically the same. Due to the lower cost of the second it was chosen for the standard abrasive.

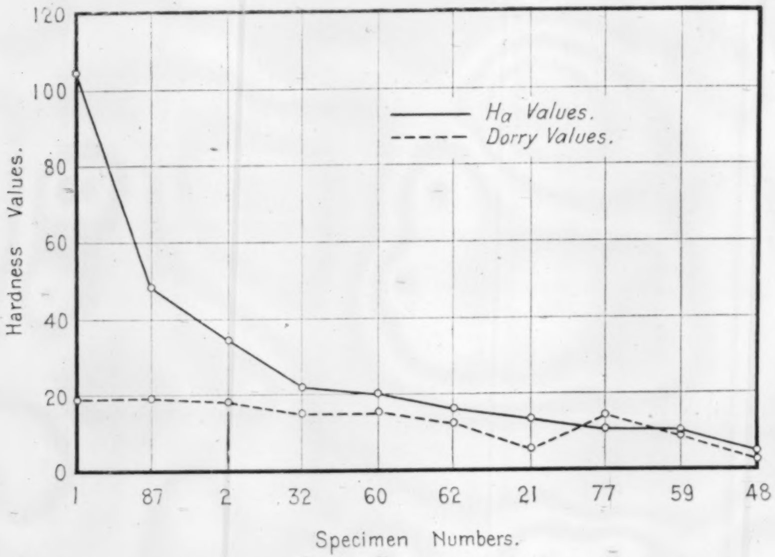


FIG. 4.—Comparison of Results Obtained with the Apparatus and the Dorry Apparatus on Ten Stones.

TEST PROCEDURE

A standard procedure was then established for the tests as follows: The specimens, loaded with two kilograms, were abraded for 225 revolutions of the disk with a feed of No. 60 artificial corundum through a $\frac{3}{32}$-in. circular orifice spaced $\frac{3}{8}$ in. above the disk and $1\frac{1}{8}$ in. from the center. The time required for testing three specimens was 5 minutes, during which about $\frac{1}{3}$ lb. of abrasive was used. Since it was found that high humidity affected the results[1] the tests were made when the humidity was between 30 and 50. The abrasion loss, W_a, was determined by weighing to the nearest 0.01 g. which was reduced to its original volume in cubic centimeters by dividing by the apparent specific gravity, G. In order to obtain values of the same order, as by the Moh's scale, the result H_a was expressed as a reciprocal of the volume loss. This reciprocal was arbitrarily multiplied by 10 in order to avoid fractional values for soft materials. The reduction formula was as follows:

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[1] Serpentine is here classed as a marble since it is ordinarily used as such.

$$H_a = \frac{10}{\text{corrected volume loss}} = \frac{10}{\dfrac{2000}{2000 + W_s} \times \dfrac{W_a}{G}} = \frac{10\,(2000 + W_s)\,G}{2000W_a}$$

where W_s is the original weight of the specimen and the other symbols are as indicated above. The correction for the weight of specimen as given in this formula was checked by several experiments and found to be reliable.

A comparison of results obtained with this apparatus and the Dorry apparatus on 10 materials is shown in Fig. 4. The great difference for the hard materials is due to the difference in methods of expressing the results. As previously shown by the formula used for the Dorry results no material could have a hardness greater than 20. The above formula for H_a values has the advantage of expressing the hardness of materials in their true relation to each other; that is, if two materials have H_a values of 4 and 8, respectively, it is known that the second is abraded twice as fast as the first.

RESULTS OF TESTS

Tests have been made on 47 marbles, 15 limestones, 4 sandstones, and a few miscellaneous materials. The results are given in Table I.

It will be noted that the abrasive hardness values of certain materials as given are at variance with our usual conception of hardness. Hard wood, linoleum and rubber tile possess a high resistance to abrasion and take a position among very hard materials. This points clearly to the fact that toughness is an important factor in determining the wear resistance. It is probably for this reason that some of the values are out of proportion to those assigned for the

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[1] Same effect noted by A. C. Harrison, "A Study of the Resistance to Abrasion of Ceramic Glazes," *Journal*, Am. Ceramic Soc., February, 1927, Vol. 10, No. 2.

TABLE I.—ABRASIVE HARDNESS RESULTS.

MARBLES

SERIAL NUMBER	MATERIAL	H_a VALUES	SERIAL NUMBER	MATERIAL	H_a VALUES
1-M_1	Vermont Serpentine[a] (Roxbury, Vt.)	104.0	24	Carthage (Missouri)	14.4
2	Oriental (Swanton, Vt.)	34.3	25	St. Genevieve (Missouri)	17.8
3	Champlain Black (Vermont)	24.6	26	Colorado Yule (Colorado)	6.7
4	Brandon (Vermont)	15.4	27	Chocolate (Tennessee)	18.1
5	Rutland (Vermont)	11.5	28	Amaico (Tennessee)	18.5
6	Danby (Vermont)	9.9	29	Cumberland Pink (Tennessee)	19.4
7	Dorset Gray (Vermont)	12.8	30	Cumberland Gray (Tennessee)	22.6
8	Florentine Blue (Vermont)	12.8	31-M_4	Phoenix Pink (Tennessee)	19.3
9	Hollister (Vermont)	14.1	32	Victoria Pink (Tennessee)	22.0
10	Albertson Blue (Vermont)	14.9	33	Batesville (Arkansas)	21.8
11-M_2	Mountain White (Vermont)	9.8	34-M_8	Pink Kasota (Minnesota)	15.8
12	Brocadillo (Vermont)	6.7	35	Vert Tinos[b] (Greece)	78.3
13	Venoso (Vermont)	12.1	36	Blanco P (Italy)	13.9
14	Nashobe Gray (Vermont)	9.1	37	English Vein (Italy)	21.4
15	Gravina (Alaska)	15.7	38-M_6	Tavernelle Fleuri (Italy)	34.5
16	Mezzotint (Georgia)	14.4	39	Botticino (Italy)	32.6
17	Etowah (Georgia)	18.3	40	Black and Gold (Italy)	25.4
18	Cherokee (Georgia)	11.8	41	Levanto (Italy)	17.3
19	Lee (Massachusetts)	14.4	42-M_9	Ordinary Italian (Italy)	17.5
20	York Fossil (New York)	25.6	43-M_7	Roman Travertine	15.7
21	South Dover (New York)	13.0	44-M_{10}	Hauteville (France)	32.1
22	Regal Blue (North Carolina)	26.3	45	Grand Antique (France)	29.6
23-M_5	Alabama Grade A (Alabama)	14.8	46	Champville (France)	17.0
			47-M_3	Belgian Black (Belgium)	29.9

LIMESTONES

SERIAL NUMBER	MATERIAL	H_a VALUES	SERIAL NUMBER	MATERIAL	H_a VALUES
48-L_4	Bedford Statuary (Buff No. 852)	3.9	54	Bedford Special Hard (No. 231)	6.9
49-L_1	Bedford Grade A (Buff No. 869)	6.2	55	Stinesville (Buff No. 93)	6.2
50-L_5	Bedford Grade A (Buff No. 870)	6.2	56	Stinesville (Buff No. 94)	6.9
51-L_6	Bedford Grade B (Buff No. 876)	8.0	57	St. Paul, Ind	23.4
52-L_9	Bedford Grade B (Buff No. 881)	6.3	58-L_3	Bowling Green (Kentucky)	3.9
53-L_2	Bedford Grade C (Buff No. 888)	5.1	59-L_{11}	Rockwood (Alabama)	8.7
			60-L_{10}	Onondaga (New York)	19.4
			61-L_8	Mankato Gray (Minnesota)	8.8
			62-L_7	Kasota Pink (Minnesota)	15.8

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[a] Serpentine is here classed as a marble since it is ordinarily used as such.

[b] A "Verde Antique Marble."

TABLE I.—ABRASIVE HARDNESS RESULTS.—*Continued.*

SANDSTONES

SERIAL NUMBER	MATERIAL	H_a VALUES	SERIAL NUMBER	MATERIAL	H_a VALUES
63	Birmingham Buff (Ohio)	7.9	65	Briar Hill L. V. (Ohio)	2.6
64	Briar Hill G. B. (Ohio)	2.5	66	Aquia Creek (Virginia)	7.3

SLATES

SERIAL NUMBER	MATERIAL	H_a VALUES	SERIAL NUMBER	MATERIAL	H_a VALUES
67-S_1	Bangor (Pennsylvania)	7.6	74-S_8	Bangor (Pennsylvania)	7.4
68-S_2	Pen Argyl (Pennsylvania)	6.6	75-S_9	Pen Argyl (Pennsylvania)	7.5
69-S_3	Wind Gap (Pennsylvania)	6.5	76-S_9	Bangor (Pennsylvania)	8.2
70-S_4	North Bangor (Pennsylvania)	7.1	77	Chapman (Pennsylvania)	10.5
71-S_5	North Bangor (Pennsylvania)	6.4	78	Purple Slate (Vermont)	7.3
72-S_6	Wind Gap (Pennsylvania)	7.0	79	Fairmont (Georgia)	9.6
73-S_7	Bangor (Pennsylvania)	7.8	80	Granville Red (New York)	12.5

MISCELLANEOUS MATERIALS

SERIAL NUMBER	MATERIAL	H_a VALUES	SERIAL NUMBER	MATERIAL	H_a VALUES
81-*A*	Soapstone (Alberene)	10.2	87	Granite	48.0
82	Onondaga Litholite	14.0	88	White Pine	35.0
83	Anhydrite	8.5	89	Seasoned Birch	96.0
84	Gypsum	7.1	90	Rubber Tile	133.0
85	Magnesite	43.4	91	Linoleum	180.0
86	Quartz	156.0			

same materials by the Moh's scale. For instance, treatises on mineralogy give the hardness of serpentine as ranging between 2.5 and 4 which is very near the values assigned for marble. The H_a values given in the table for materials Nos. 1 and 35, which are serpentines, indicate a hardness three to fifteen times that of marble. Likewise gypsum, which is rated as a soft rock, is shown to have an H_a value greater than several of the limestones.

SERVICE TESTS

The scheme used for comparing the abrasion results with the wear on floors consisted in measuring the wear on 84 tiles of marble, slate and limestone placed on the treads of a narrow stairway and testing the same materials on the abrasion apparatus. The tiles were made of a size, namely, 4 in. square and $\frac{13}{16}$ in. thick, that they could be weighed to some degree of precision. A special form was made for holding the tiles in place on the treads, which consisted mainly of wood with a composition board at the bottom for the tiles to rest

FIRST STEP

M_2-4 0.14	L_3-5 0.31	M_2-6 0.53	S_9-10 0.62	L_9-11 0.49	S_{12}-12 0.14
L_3-1 0.70	M_3-2 0.20	L_3-2 1.63	L_9-7 0.98	S_3-8 0.50	L_9-9 0.24

SECOND STEP

L_4-4 0.28	S_4-5 0.34	L_4-6 0.61	L_{11}-10 1.11	L_{10}-11 0.06	L_{11}-12 0.10
S_1-1 0.31	L_4-2 1.01	S_1-3 0.61	L_{10}-7 0.32	L_{11}-8 0.49	L_{10}-9 0.11

THIRD STEP

L_2-4 0.23	S_2-5 0.18	L_2-6 0.57	M_7-10 0.20	L_8-11 0.16	M_7-12 0.02
S_{11}-1 0.50	L_2-2 1.38	S_{11}-3 0.51	L_8-7 0.71	M_7-8 0.34	L_8-9 0.25

FOURTH STEP

M_1-4 0.15	L_1-5 0.41	M_1-6 0.16	L_7-10 0.16	L_5-11 0.18	L_7-12 0.11
L_1-1 1.18	M_1-2 0.29	L_1-3 2.11	L_5-7 0.96	L_7-8 0.44	L_5-9 0.36

FIFTH STEP

A-4 0.11	L_6-5 0.30	A-6 0.32	M_5-10 0.06	S_4-11 0.18	M_8-12 0.00
L_6-1 0.30	A-2 0.78	L_6-3 0.91	S_5-7 0.71	M_8-8 0.28	S_5-9 0.21

SIXTH STEP

M_6-4 0.02	S_7-5 0.16	M_6-6 0.04	M_3-10 0.04	M_{10}-11 0.03	M_3-12 0.03
S_{10}-1 0.35	M_6-2 0.14	S_{10}-3 0.54	M_{10}-7 0.18	M_3-8 0.23	M_{10}-9 0.06

SEVENTH STEP

M_4-4 0.02	M_5-5 0.05	M_4-6 0.06	S_6-10 0.15	M_9-11 0.05	M_3-12 0.03
M_5-1 0.14	M_4-2 0.11	M_5-3 0.30	M_9-7 0.43	S_2-8 0.55	M_9-8 0.12

(*a*) First service test, 2 months' exposure.

FIRST STEP

L_4-4 0.14	M_7-5 0.17	M_5-6 0.29	L_6-10 0.35	S_5-11 0.17	L_3-12 0.10
M_9-1 0.07	M_5-2 0.12	L_{10}-3 0.18	M_4-7 0.18	M_9-8 0.17	M_1-9 0.00

SECOND STEP

S_{10}-4 0.04	L_2-5 0.25	M_2-6 0.11	S_4-10 0.16	L_9-11 0.08	S_4-12 0.05
M_9-1 0.11	M_4-2 0.14	L_{11}-3 0.40	M_3-7 0.18	M_9-8 0.14	M_{10}-9 0.00

THIRD STEP

L_9-4 0.12	L_5-5 0.14	L_6-6 0.12	M_5-10 0.15	L_4-11 0.15	M_6-12 0.00
M_9-1 0.07	L_5-2 0.48	M_7-3 0.27	M_6-7 0.11	M_9-8 0.16	L_4-9 0.26

FOURTH STEP

M_9-4 0.02	M_9-5 0.04	M_9-6 0.06	M_9-10 0.04	M_9-11 0.04	M_9-12 0.02
M_9-1 0.10	M_9-2 0.19	M_9-3 0.27	M_9-7 0.27	M_9-8 0.14	M_9-9 0.08

FIFTH STEP

A-4 0.07	L_{10}-5 0.08	L_{11}-6 0.10	A-10 0.17	M_6-11 0.06	L_6-12 0.02
M_9-1 0.08	M_3-2 0.12	L_2-3 0.91	M_1-7 0.11	M_9-8 0.15	S_{11}-9 0.20

SIXTH STEP

M_{10}-4 0.02	S_{10}-5 0.06	M_2-6 0.10	S_{11}-10 0.16	L_3-11 0.24	S_3-12 0.04
M_9-1 0.06	L_2-2 0.58	L_{10}-3 0.17	M_5-7 0.30	M_9-8 0.14	M_1-9 0.04

SEVENTH STEP

L_9-4 0.04	L_{11}-5 0.02	S_9-6 0.00	S_5-10 0.01	M_4-11 0.02	L_3-12 0.05
M_9-1 0.03	A-2 0.12	M_{10}-3 0.07	L_5-7 0.42	M_9-8 0.12	M_7-9 0.04

(*b*) Second service test, 1 month's exposure.

FIG. 5.—Results of Service Tests on Stair Treads.

on and a brass angle at the front. The tiles were wedged in place in the recess of the form by means of strips of cardboard which also served to separate the tiles from each other and from the metal of the form. The first set of measurements was made on the set of tiles shown in Fig. 5(*a*) for a service period of two months. Although the traffic on this stairway is rather light, practically all of the tiles showed a very definite amount of wear. The materials are designated by letters M, S, L and A representing marble, slate, limestone, and alberene, respectively. The sub-figures indicate the particular materials, which may be identified by reference to Table I. The numbers entered below the letter designations in Fig. 5(*a*) indicate

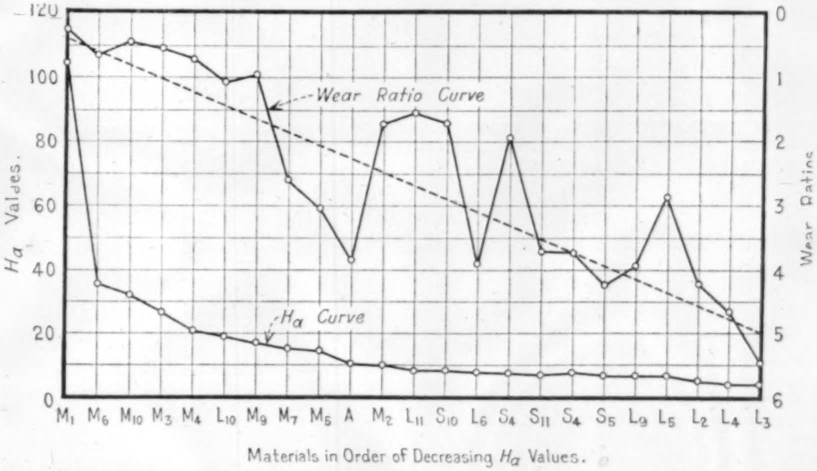

FIG. 6.—Comparison of H_a Values for Twenty-three Materials with the Results of Service Tests.

the volume wear in cubic centimeters which were determined in the same manner as in the abrasion tests.

These measurements afford some information of interest but because of the fact that the wear varies greatly in different parts of the tread this arrangement permitted no way of correlating the results on different materials. Hence another arrangement of the tiles was made as shown in Fig. 5(*b*) and submitted to one month of service. It will be noted that the fourth step was tiled only with M_9. By placing one material in all positions on one step a comparison of the wear on this material was obtained with all others for the various positions. Assuming that a given position on one step is submitted to the same amount of wear as that position on any other step, the wear on tiles in position No. 3, for instance, will indicate the relative hardness of the materials in this position. In order to test the accuracy of this assumption the material M_9 was also placed in positions 1 and 8 on all the steps as shown in Fig. 5(*b*). It will be noted that the wear indicated for these positions is reasonably constant except on the seventh step where it was much less than on any of the others; hence this step was disregarded in computing the results. In arriving at the relative wearing qualities of the various materials from this experiment the ratio of the wear of each to that on M_9 for the different positions was computed. In Fig. 6 the various materials employed in this experiment have been arranged in order of their H_a values and the wear ratios obtained from the tread measurements plotted to the scale on the right. The broken line was drawn to show the average trend of wear ratios.

DISCUSSION OF RESULTS

While it may be said that the average trend of the wear ratios agrees with the H_a values, several materials show discordant results. This is believed to be due in part to the fact that the period of the service test was too short and for that reason the wear was not great enough in some cases to be definite. It may also be possible that some materials possess certain properties by virtue of which they resist the conditions of mechanical wear better than those of service wear or *vice versa*. Material M_1 which showed a very high H_a value did not resist service conditions much better than M_6 which appeared from the abrasion test to be only about one-third as hard. This was apparently due to the fact that M_1 had veins of soft material which were worn faster under service conditions. Materials M_{10}, M_3, M_4, L_{10}, and M_9 were very much alike in being dense, fine-grained materials of high strength. Material M_7, the first to drop below the average line of wear ratios, being a travertine, it might be expected to crumble to some extent under the impact of traffic. M_5 is very similar to some of the marbles that took a higher position in the service tests and no plausible reason can be offered for its low wear ratio. The soapstone *A* showed a more discordant result in the two tests than any other material, which is believed to be due to its inability to resist the impact of traffic. This was indicated by the tile wearing to a rough surface. M_2, L_{11}, S_{10}, S_4, and L_5 would receive a considerably higher rating from the service tests than for the abrasion test, while with the remaining tests the two methods are in fair agreement. Both methods point to the conclusion that the marbles and dense limestones possess the best wearing qualities, that the slates are somewhat less resistant and the oolitic limestones were the least resistant of the materials in the service tests.

CONCLUSIONS

1. Comparison of abrasion test results with results of service tests on identical materials shows a fair agreement and indicates that the testing apparatus gives valuable information as to wearing qualities.

2. Some evidence was developed that certain properties of materials other than abrasive hardness may influence the wearing qualities under service conditions. Toughness and strength as well as texture variations appear to be important considerations.

3. Humidity affects the abrasive action; that is, tests made during periods of high humidity indicate lower abrasive hardness values than tests made when the humidity is low.

4. Abrasion tests as well as actual wear measurements on 9 marbles, 8 limestones, and 5 slates indicate that marble is the most resistant, slate next, and limestone least. The denser limestones included in these tests show wear resistance values more like those of marble.

DISCUSSION

Mr. Boyd. Mr. D. Knickerbacker Boyd.[1]—As an architect I am very much interested in this question of the wearing of floor and stair materials. We have not yet, it seems to me, had the solution offered of how best to determine the appropriate use of materials in floors for given locations in a building or in certain classes of buildings. This paper is an excellent step in the right direction, but we have as yet no means for determining the length of duration or wear under a given use, and I am wondering whether Mr. Kessler or some other investigator has considered the subject of trying alternately to lift and drop the specimen while the wheel is rotating so that the results could be offered in terms of probable expectancy of wear. For instance, if he grinds down, say, $\frac{1}{4}$ in. or $\frac{1}{16}$ in. on any given material, could he furnish us any information as to how long it would take under normal use and actual conditions of wear to accomplish the same results in, say, the ground floor corridors of a department store or the lobby of a hotel, or the concourse in a railroad station. And, conversely, where the same materials would not be subject to much wear and where there would not be much abrasive matter brought in on the feet of pedestrians, could we get some idea of how such a floor would wear over a long period of years—take, for instance, a corridor of an upper floor on which people presumably walk only with clean leather or rubber-soled shoes. In other words, is there some method by which this wearing away can be resolved into periods of time according to location and use instead of mere measurements of wear?

Perhaps the best way to do that would be to put down several different kinds of materials in a certain location of a building where all were subjected to fairly constant wear, selecting materials of different hardness, and then making some kind of determinations, about as Mr. Kessler did on the stairway referred to, by actual weight and measurements, to establish means of comparing the wear on them, and in that way form more specific ideas as to how long materials might last on floors under each condition.

There is another factor to be considered in connection with the subject—the question of slipperiness. It might be possible under this same series of tests to show that a certain material would practically not wear away under given use, but yet might become slippery and

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[1] Architect and Structural Standardist, Structural Service Bureau, Philadelphia, Pa.

Mr. Boyd.

therefore be unavailable, or should not be selected, for that particular use or location in a building. And still another thought is that a material might be so little liable to wear itself that it might wear down everything that trod on it. I have in mind, for instance, school houses where so much thought could be given to not having the steps wear down that, covered with a material which would wear the soles of the shoes of all the children, they might be so effective in that, that it would be better if they were replaced by a softer material which would itself wear more readily.

Mr. Cartwright.

MR. F. P. CARTWRIGHT.[1]—There are two questions I should like to ask. The first has to do with whether the influence of the different commercial cleaning fluids was considered. It has been found in certain public buildings, particularly in school buildings, that the use of commercial cleaning fluids at times in rather large amounts has greatly influenced the wear of floors.

The second question I should like to ask is, how much or to what extent do check runs differ, using this grit with the material of the same density or hardness? In other words, what is the degree of tolerance in results with the same material?

Mr. Kessler.

MR. D. W. KESSLER.[2]—With regard to the first question, we know fairly well that cleansing materials do affect the wear of floors. In the case of materials containing grit (volcanic ash is one of the common grits used in these), if a small amount is left on the floor it is ground under foot traffic and increases the rate of wear. There is another factor that probably is not often given consideration and that is the salt content of cleansing materials. If such preparations are not properly used, the salts may penetrate a porous floor and crystallize therein, giving the same action as efflorescence does on the outside of buildings. In that case it is not exactly wear but disintegration of the floor itself. It would seem that some floors go to pieces much more rapidly from that cause than from actual wear.

With regard to the checks in different tests, that depends a good deal on the material being tested. One will find in some marbles that the variation may be 10 per cent or even more, but I think that is due more to the variation in hardness of the marble itself. Marble, as you know, contains different minerals—very often quartz or flints—which are very much harder than the calcium carbonate of marble, and if a test piece contains one of those crystals the piece will show up much harder than some other specimens. With very uniform materials the checks may run as close as 1 per cent, but I should say as an

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[1] Chief Engineer, National Lumber Manufacturers' Association, Washington, D. C.

[2] Civil Engineer, U. S. Bureau of Standards, Washington, D. C.

Mr. Kessler. average for all materials we should expect a variation of about 5 per cent.

The Secretary-Treasurer. THE SECRETARY-TREASURER (*Mr. C. L. Warwick*).—I should like to point out one thing in connection with this subject. The program as submitted this year schedules a discussion on Wear Testing of Metals, and I think there are some points that are brought out in the papers of that discussion and in the written and oral discussion of the papers that will be of interest to those concerned with this particular aspect of wear testing.

Mr. Kern. MR. LEROY E. KERN.[1]—I understand that during the World War there was a machine devised for testing the wearing qualities of different kinds of soles of shoes which were to be furnished to the Army. That device was later taken over by the Post Office Department to try and ascertain what kinds of soles would last the longest on the postmen making deliveries throughout the United States, and I am wondering if Mr. Kessler considered the possibility of taking that machine and reversing it so as to show the wear on the object on which the shoe trod instead of measuring the wear on the shoe. That is the sort of information an architect wants, not only under abrasive conditions but under normal wear of leather soles, for instance.

Mr. Kessler. MR. KESSLER.—That point has been considered but it would be very hard to keep a leather sole or whatever material one were using in a uniform condition, because it wears so much faster than the floor itself, and one would need to be continually changing the medium. I doubt whether one would be able to establish any uniform conditions with leather.

I might refer to the work of Mr. Hunter of the U. S. Bureau of Standards. In trying to determine the slipperiness of different flooring materials he uses a leather sole and measures the friction of that against the different materials. He found that the humidity of the air affects the leather to such an extent that by testing the same sample of flooring on different days he may get a variation in his results of 100 per cent. We would, perhaps, meet with that same difficulty in trying to test flooring materials with a material like leather.

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[1] Technical Secretary, Structural Service Department, Am. Inst. of Architects, New York City.

AN EXPLANATION OF SOME OF THE DIFFICULTIES IN ABRASION TESTING OF RUBBER[1]

BY HARLAN A. DEPEW[2]

SYNOPSIS

The paper discusses abrasion resistance of rubber, pointing out that the author's tests have indicated that abrasion resistance is not a fundamental property but is rather the resultant of resistance to cutting and tearing.

An abrasion machine used at the beginning of the investigation gave satisfactory relative abrasion resistance on many rubber stocks, but not on unusually hard or soft stock. Examination of the abraded surfaces showed that the test was largely a cutting test. A modified machine was then constructed which made the test largely a tearing test, and abrasion resistance as determined on the modified machine is more nearly in accord with actual experience with rubber of all cures.

The paper presents certain abrasion test data obtained with the two machines, and modifications are described by which the test can be made more of a cutting test or a tearing test as specific needs may require.

The purpose of this paper is to point out that abrasion resistance is not a fundamental property of a rubber stock, but rather that what we call abrasion resistance is the resultant of resistance to cutting and tearing, which are far from constant in relative importance. It is quite probable that as this fact becomes more generally accepted, abrasion tests, as now reported, will be discontinued, and that instead cutting resistance and tear resistance will be reported. Then the resistance to abrasion of rubber products in any kind of service can be calculated by properly weighting these factors.

Whether the cutting test generally used will be a knife test and whether the tear tests will be of the tensile type cannot be told at the present time; but it is quite possible that some kind of abrasive wheel methods will be used to determine cutting and tearing resistance, because tests of this sort give an average of a large number of small cuts or a large number of small tears as the case may be. Tear resistance in turn can probably be sub-divided into several kinds of tear resistance depending on the size and rate of application of the tearing forces. This would involve a big research program in itself, and will not be discussed further in this paper.

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[1] The same factors that determine the abrasion resistance of rubber probably also apply to most other materials.

[2] Research Division, New Jersey Zinc Co., Palmerton, Pa.

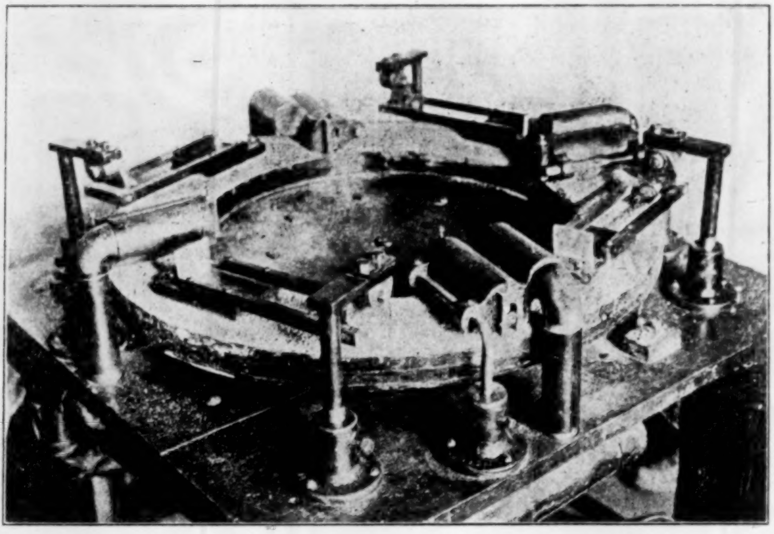
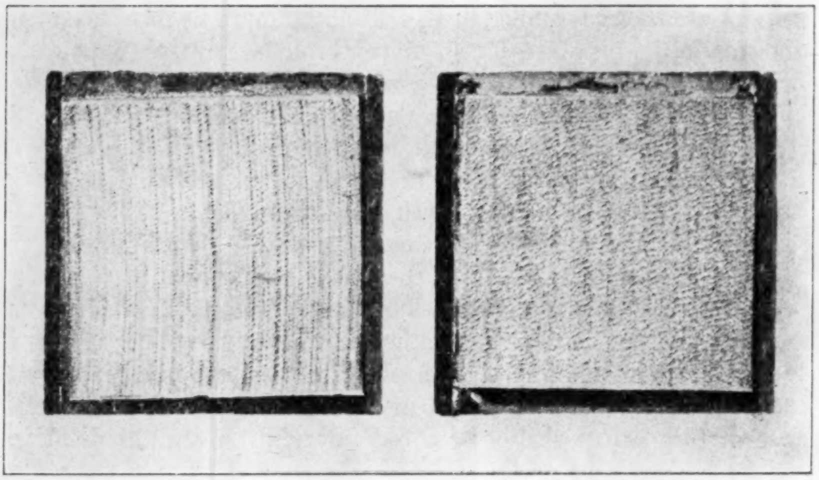


FIG. 1.—First Design of the Abrasion Machine.



(a) Tested without lifts. (b) Tested using lifts.

FIG. 2.—Comparison of the Abraded Surfaces as Determined on the Abrasion Machine Without and With Lifts.

In the series of tests that led up to the conclusion that abrasion resistance was the resultant of cutting and tearing resistance, an abrasion machine was used, in the beginning, consisting of an arrangement whereby four rubber test pieces were held against a rotating concrete track with a definite force and the relative volume losses of these compounds were determined, the abrasion resistances being inversely proportional to the volume losses.

One of the four test pieces is a standard, the 75-minute press cure at 141.5° C. of the compound referred to in Table I being chosen. The abrasion resistance of this standard became 115 as an outgrowth of laboratory development of a new standard of greater duplicability, the original standard having been set arbitrarily at 100. (If another more duplicable standard should be developed with an abrasion resistance of 165, for example, it would be adopted with that value, so that old and new results would continue to be comparable.)

TABLE I.—TENSILE PROPERTIES OF A RUBBER COMPOUND OVER A RANGE OF CURES AND THE ABRASION RESISTANCE UNDER DIFFERENT CONDITIONS OF TESTING.

Compound: 20 volumes of XX Red zinc oxide to 100 volumes of rubber, 6 per cent of sulfur and 1 per cent Hexa by weight calculated on the rubber content.

Time of Cure at 141.5° C., min.	Tensile Strength, lb. per sq. in.	Elongation, per cent	Stress at 300 per cent Elongation, lb. per sq. in.	Stress at 450 per cent Elongation, lb. per sq. in.	Cutting Abrasion Test	Tearing Abrasion Test
15	1955	625	425	970	41	79
30	2695	625	555	1310	64	93
60	3420	605	820	1745	101	112
75	3735	620	935	1930	115	115
90	3685	605	945	1975	128	102
105	3715	595	1065	2105	131	106
120	3475	575	1125	2150	141	108
150	3205	535	1195	2230	140	71
180	2735	510	1245	2205	119	60

A view of this machine after preliminary refinements is shown in Fig. 1. It was soon found that although this machine gave satisfactory relative abrasion resistance on many stocks, it did not give the kind of results on unusually hard or soft stocks that we expected from tire service. A hard stock gave too high results and a soft stock gave too low results. This indicated that the test was largely a cutting test since hardness of materials is usually associated with resistance to cutting. An examination of the abraded surface in Fig. 2 (*a*) tends to confirm this theory in that it showed cutting lines made by the cutting edges of the track abrasive. We note this kind of wear on an automobile tire that is spinning on slippery roads, but the normal wear appears to be irregular and consists in the tearing out of small pieces of rubber.

Accordingly, in our development of an abrasion machine to simulate tire service, modifications were tried out to increase the tearing and to reduce the cutting by means of a system of lifts which raise the test-piece holders and drop them on the track. The appearance of

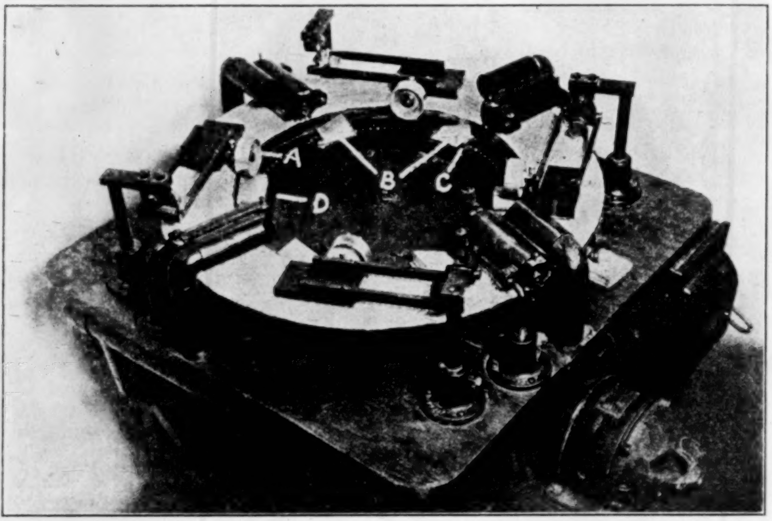


FIG. 3.—Top View of the Modified Abrasion Machine Equipped with Six Lifts.

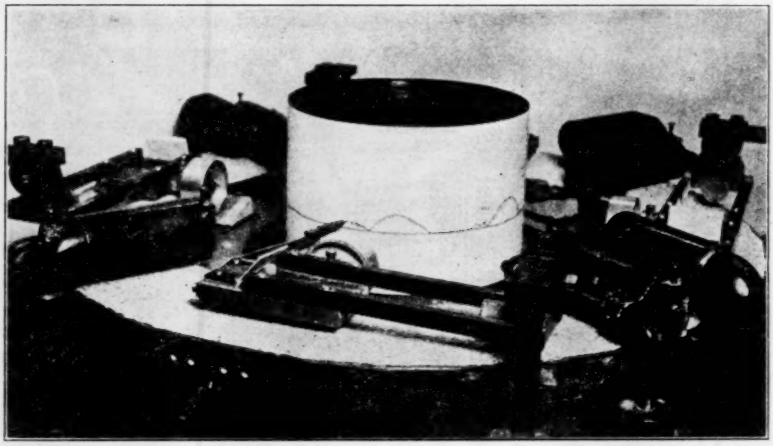


FIG. 4.—Autographic Record of the Motion Given to a Test Piece by the Lifts.

the abraded surface when tested on this machine is radically changed as shown in Fig. 2 (*b*). Little particles of rubber were torn out here and there at random.

The design of the modified machine is shown in Fig. 3 and the motion of the holder is shown by the autographic record, taken as shown in Fig. 4. The curve shows that the test-piece holder bounds from the track after each drop and then strikes it again. Using this machine the abrasion resistance no longer increases with increasing cure but reaches an optimum at about the same cure as in a tension determination, which is in part a tearing test.

TABLE II.—COMPARISON OF THE EFFECT OF AGING ON ABRASION RESISTANCE AS DETERMINED WITH AND WITHOUT LIFTS.

Time of Cure at 141.5° C., min.	Abrasion Resistance Without Lifts				Abrasion Resistance With Six Lifts			
	No Aging	Aged 4 days at 70° C.	Aged 7 days at 70° C.	Aged 14 days at 70° C.	No Aging	Aged 4 days at 70° C.	Aged 7 days at 70° C.	Aged 14 days at 70° C.
15........	41	41	40	35	79	80	61	66
30........	64	86	56	45	93	82	73	74
60........	101	92	81	67	112	87	77	62
75........	115	109	93	78	115	115	67	61
90........	128	117	106	85	102	100	58	54
105........	131	114	101	85	106	82	52	48
120........	141	111	93	83	108	69	47	43
150........	140	109	91	93	71	55	33	35
180........	119	99	81	95	60	43	23	30

TABLE III.—COMPARISON OF RESULTS OF THE NEW JERSEY ZINC CO. ABRASION MACHINE EQUIPPED WITH SIX LIFTS WITH THE RESULTS FROM THE U. S. MACHINE.

Time of Cure at 141.5° C., min.	Abrasion Resistance	
	New Jersey Zinc Co. Machine	U. S. Machine
15........	79	106[a]
30........	93	89
60........	112	111
75........	115	115
90........	102	110
105........	106	112
120........	108	103
150........	71	99
180........	60	88

[a] A decided undercure gives slightly high results due to the tacky stock balling up.

Table I shows that the tensile strength optimum comes at the 75-minute cure and that the tearing abrasion resistance (machine with six lifts, Fig. 3) also reaches an optimum at the 75-minute cure, whereas, the cutting abrasion resistance (machine without lifts, Fig. 1) reaches an optimum at the 120 to 150-minute cure. The same effect is shown in Table II, where aged test slabs have been abraded on the two modifications of this machine. The resistance to tear steadily decreases on aging and the overcures are affected more than the undercures. The abrasion optimum that came at the 75-minute cure on the original, comes at the 30-minute cure after 14 days in the Geer oven. In the case of the machine without lifts, the optimum is somewhat irregular due to the fact that the resistance to cutting is influenced by the hardness, and the stock tended to soften first on aging and then to harden.

Most of the abrasion machines that are now in service are largely tearing tests due either to the choice of abrasive or to the construction of the machine. Accordingly, the machine without lifts does not give results comparable with other machines, but when equipped with six lifts it does give similar results. Table III gives a comparison of the results obtained on the machine equipped with six lifts and on the U. S. machine, the close check results being typical of the results on a number of other abrasion machines in use to-day.

TABLE IV.—THE INFLUENCE OF MODIFICATIONS OF THE ABRASION MACHINE ON THE RELATIVE ABRASION RESISTANCE OVER A RANGE OF CURES.

Time of Cure at 141.5° C., min.	Abrasion Resistance					
	Number of Lifts	0	6	0	6	1[a]
	Track Speed, r. p. m	28	28	28	28	115
	Weight of Holders, lb	3	3	1.3	1.3	3
15		41	79	36	67	89
30		64	93	53	90	108
60		101	112	100	107	**123**
75		115	**115**	115	115	115
90		128	102	138	126	110
105		131	106	138	**135**	111
120		**141**	108	**160**	120	92
150		140	71	158	105	71
180		119	60	142	79	38

NOTE.—In each case the abrasion resistance of the 75-minute cure has been arbitrarily set at 115.

[a] Only one lift can be used at this high speed. (Peripheral Speed of track is approximately 15 ft. per sec.) If more lifts are used, the holder bounds from lift to lift without hitting the track.

The two recent papers on abrasion testing by Williams[1] and Vogt[2] have centered attention on the relation between abrasion resistance and power loss. Williams stated that it was desirable to compare stocks under the same power loss.

If we compare a first and a third grade tread on a machine of the type shown in Fig. 1, the third grade tread (which owes its abrasion resistance largely to its hardness and, therefore, cutting resistance) will absorb less power than the first grade tread (which owes its abrasion resistance to tear resistance), since very little power is required in a cutting test and since there is little difference in power required by different stocks; the cutting edge simply cutting deeper in an easily cut stock.

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[1] *Industrial and Engineering Chemistry*, Vol. 19, p. 674 (1927).

[2] *Industrial and Engineering Chemistry*, Vol. 20, p. 302 (1928).

Accordingly, if we compare rubber slabs for abrasion resistance at equal power loss, the stock with the low-power loss must be abraded under conditions of increased severity and we will have weighted tearing resistance as of relatively greater importance.

This method of increasing the relative severity of the tearing action in abrasion might be classified as a mathematical method, whereas, the method of modifying the abrasion testing machine might be classed as mechanical.

Two modifications of this abrasion machine have been discussed, but there are many other modifications that could be used. Table IV shows the influence of some of these changes and from these data we can conclude that the percentage of abrasive wear due to tearing may be increased by (1) increasing the number of lifts, (2) increasing the speed of the track, and (3) increasing the weight of the holder. There is a limitation to the number of lifts that can be used depending somewhat on the speed of the track, because if the number is too great the test piece holder will bound from one lift to another without the test specimen even hitting the track. With the machine running at too high a speed, there are difficulties due to the fact that too much heat is generated, and if the weight of the holder is made too great there is a tendency to pound the track to pieces. In the original description of the abrasion machine it was shown that the nature of the abrasive track makes a considerable difference in the relative wear of compounds, as would have been expected in the case of a cutting test. When the machine is so designed that the test becomes largely a tearing test the nature of the track is a less important factor.

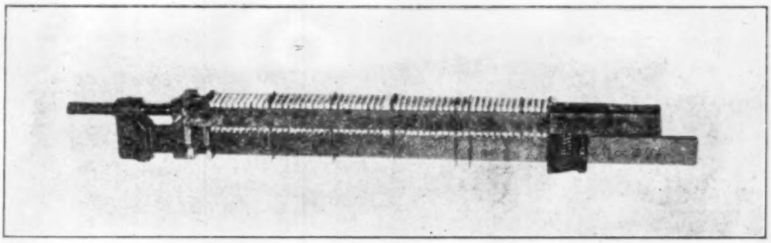

FIG. 5.—Large Clamp for Holding the Test Pieces After They Are Cemented to the Hard Rubber.

It is well known that stretched rubber, that is, under strain, cuts relatively easily and we have found that by mounting the rubber in a stretched condition the test can be made even more of a cutting test than has been shown in Table IV. It might be expected that the cutting action could be further increased by wetting the track with water, but we found practically no abrasive wear under this condition. Apparently the water lubricates the surface of the abrasive so that the rubber slides over it rather than being cut by it.

This method of test has become sufficiently valuable to our laboratory so that refinements in operation have been worked out. The test piece 2 by 2 by 0.10 to 0.25 in. has been cemented with a pure gum cement to the soft rubber surface of a grooved hard-rubber holder as shown in Fig. 2, and it is then kept for several hours before testing with a number of other test pieces in a large clamp, Fig. 5. When ready to make the test the hard-rubber holder carrying the test piece is slipped through a groove in the holder, Fig. 6.

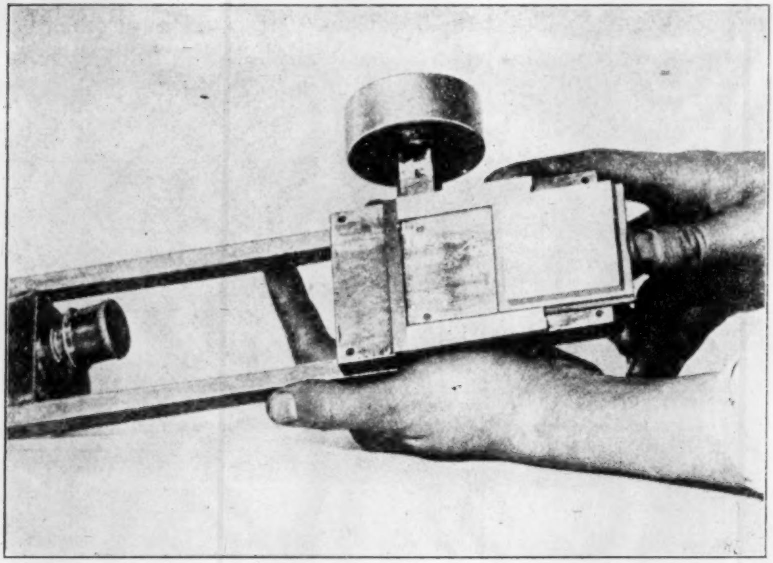

FIG. 6.—Method of Inserting Test Pieces in Holders.

The machine can be stopped, the four samples removed, new ones inserted, and the machine started again in less than one minute. Four samples, one of which is a standard, can be run for 1000 revolutions in 36 minutes. Considering the time to change test slabs and the fact that only three out of four of the test pieces are unknowns, three unknowns can be run on the machine in 37 minutes, or 12⅓ minutes to a specimen. While the machine is running, the operator can determine the specific gravities, weigh the stocks, and carry out calculations. It is important, in this work, that a good operator shall be employed who will make sure that the track is clean and who will pay attention to the innumerable details of the test.

The greatest difficulty with this method of testing is that it is a relative test and must continue to be a relative test until considerably more information is available. Accordingly, a standard is required for comparison. The criterion of this standard should be, above all other things, duplicability, and it is essential that uniform materials be used in making this standard. It is possible that we will be able to certify the standards by means of quantitative tearing and cutting tests.

If the relative tests of compounds are always the same, one standard of abrasion is sufficient. If the spread between two compounds is variable, it is desirable to have two standards, one an upper limit and the other a lower limit, and to express the unknown material in terms of these two standards.

DISCUSSION

Mr. Scott. Mr. David Scott.[1]—I should like to know if you have ever had finer grit in that machine than regular 10 mesh. In other words, can you get something near to the appearance of the abrasion of rubber in actual use through any arrangement? I have tried it and have not been able to do so.

Mr. Depew. Mr. Harlan A. Depew.[2]—I do not care whether we get a smooth surface that looks like a worn tire tread or whether we get the rough surface that I showed on the screen a few moments ago. All that I am interested in, is to obtain data that will enable us to understand the different kinds of abrasion.

Mr. Scott. Mr. Scott.—Your idea is simply a laboratory test. Can you correlate your laboratory test with the conditions of service?

Mr. Depew. Mr. Depew.—The conditions of service will be widely different. In the work that the Sub-Committee (of Committee D-11) on Abrasion Testing has been doing, we find that two different compounds give radically different results in winter and in summer, and two different compounds give radically different results when tested on one car and when tested on another. The problem is one of developing tearing and cutting tests. If we can develop tests, then we can analyze our service conditions; and from those service conditions we can say that A, who drives his car at 50 miles an hour, should have this kind of a tire tread, and B should have another kind of a tire tread.

Mr. Scott. Mr. Scott.—That is just the point that I wanted to bring out. I think that it is a thing of interest to everybody that a laboratory abrasion test is no better or no worse than any other laboratory test, and that it is of no more value simply to know the abrasive factor than it is to know the tensile strength factor. It is rather a consideration of all these tests that makes up our knowledge of compounding to suit the material for the purpose intended. Am I right?

Mr. Depew. Mr. Depew.—Yes, that is right.

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[1] Secretary and Treasurer, Henry L. Scott and Co., Providence, R. I.

[2] Research Division, New Jersey Zinc Co., Palmerton, Pa.

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